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TRANSPORTATION ENGINEERING



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INTRODUCTION

- The process of conveyance from one point to another is termed as transportation.
- Transportation has following effects over the socio-economic aspect of the life.
 - (a) It helps in progress and advancement of the community.
 - (b) Efficient transportation is essential for the economic prosperity and development of the country.
 - (c) It helps in movement - in emergency for defence of the country and to maintain better law and order.

Medium of transportation :-

- Transportation can be achieved by any of the following media.

(i) Land	} Major	(iv) Pipeline.	} MINOR.
(ii) water		(v) conveyor belt.	
(iii) Air		(vi) Elevator.	
		(vii) cable cars.	
		(viii) Rope way.	
		(ix) Hyper loop.	

On the basis of the above media of transportation following four major modes of transportation are used.

- | | |
|-----------------------|--------------------------|
| (a) Roadway / Highway | for road transportation. |
| (b) Railway | " rail " |
| (c) water way | " water " |
| (d) Airways | " Air " |

Railway :-

- It is the movement of multiple wagons or a train of wagons with steel wheels over two parallel steel rails, that offers comparatively lesser resistance. Hence the cost of transportation by this method is approx. $\frac{1}{6}$ th of that by road transportation, but less flexible.
- Railways are considered as arteries of entire transportation system.

WATER TRANSPORTATION :-

- It offers min. resistance to traction, hence is cheapest method amongst the all.
- But the time required in this case is comparatively more.
- It is suitable for transportation of bulk goods of relatively low value.

AIR TRANSPORTATION :-

- It is the fastest method available for transportation.
- But the cost involved in this is also very high.
- It is suitable for transportation of high value goods for large distance.
- This method is affected by weather conditions.

ROADWAY/HIGHWAY :-

- It is the most flexible mode of transportation amongst the all.
- But it consumes petroleum product at highest rate and rate of emission of pollution is highest in this case.
- Major road transportation is achieved by highways and expressways.
- Highways are special type of roads designed to allow high speed of vehicle.
- It is generally constructed on embankment as
 - (a) Better drainage facility
 - (b) Safety in flood time.
 - (c) no lateral entry of public or animals.

Eg:- National Highway (NH) and State Highways (SH)

- Expressways are superior type of highways which are designed as a direct source of connectivity b/w two defined points.

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- It is also known as breeeway. It organizes the traffic in channelised way.

Eg:- Kona expressway.

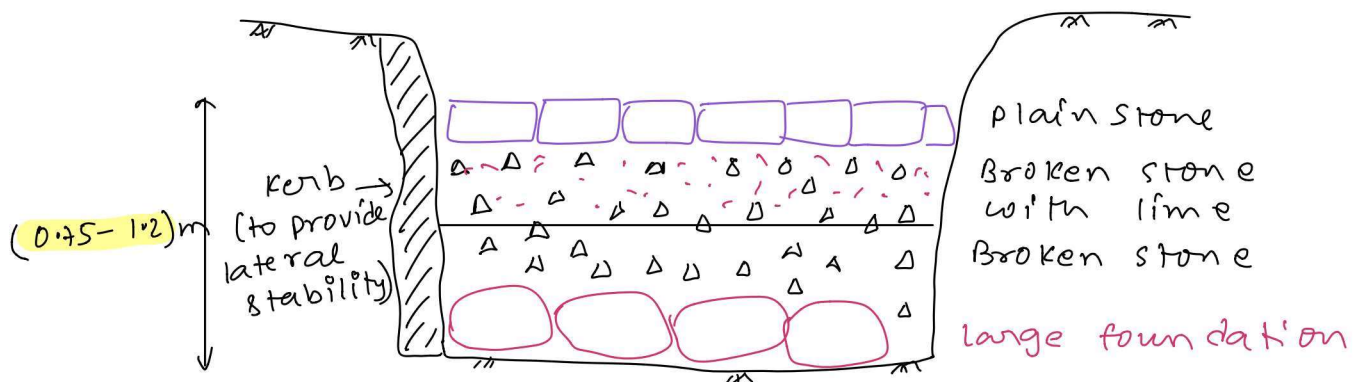
DEVELOPMENT OF ROADS :-

Development of roads took place in following sequential order.

- (i) ROMAN ROADS .
- (ii) TRESAGUET " .
- (iii) METCALF " .
- (iv) MACADUM " .

(i) ROMAN ROADS :-

- These were the earliest of the roads developed for their military purposes.
- These roads involves
 - (a) They were built straight regardless of gradient.
 - (b) The soft soil was excavated and removed up to an extent hard strata was reached.
 - (c) The total thickness of construction was in range of (0.75 - 1.2) m.



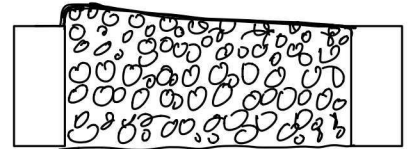
Drawbacks :-

- (i) No cross-slope is provided.
- (ii) No drainage system.
- (iii) large foundation stones were provided at bottom which are of no use, as pressure due to surface load decreases with depth. Hence they only increase the cost.

(ii) TRESAGUET ROADS :- (France)

- The main feature of these roads was reduction in overall thickness of the road upto 30cm.
- In this case due consideration was given to the moisture condition and drainage of the road.
- The subgrade was prepared and a large layer of foundation stone are laid on edges, which act as curb stone.
- The space b/w the kerb is then filled with smaller stones, sized of which reduced as we approach to top.
- Here cross-slope of 1 in 45 over the surface is also provided.

note:- Metcalc road were developed in parallel to previous one, but no recorded literature is available for it. (developed in England)



Tresaguet Road.

19th Dec

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(iii) TELFORD ROADS :-

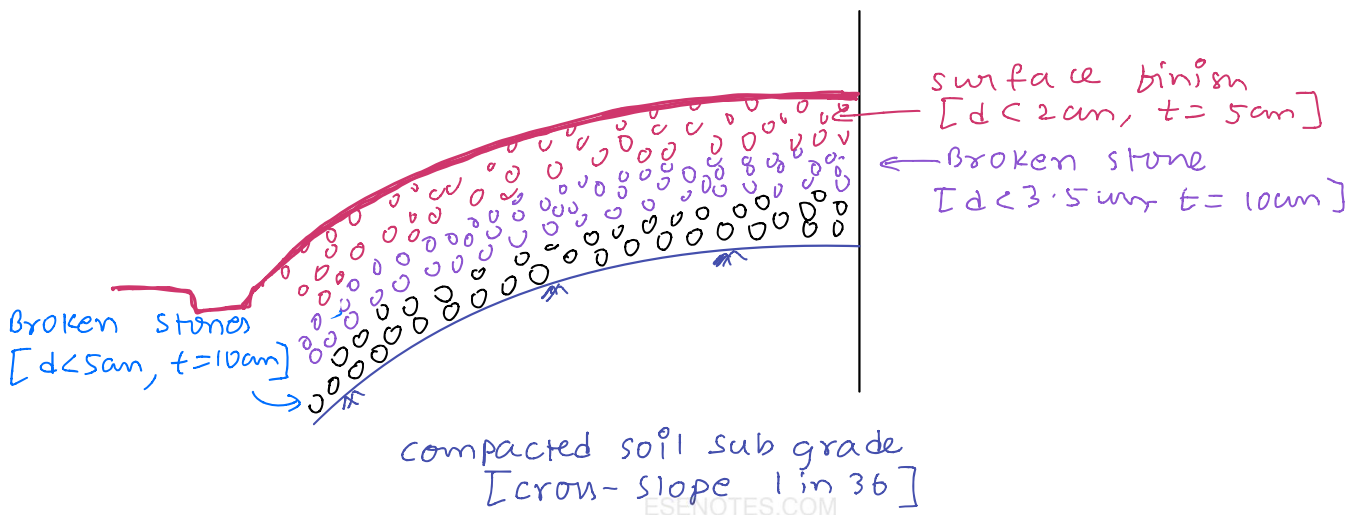
- In this also heavy/large foundation stones were provided above the soil subgrade and cross slope at top surface was given to ensure removal of the water.
- A level subgrade was prepared of width 9m
- large foundation stones of thickness (17-22)cm was laid over the subgrade, with larger stone at the centre and smaller at the edges to provide 1:45 slope
- The central portion of about 5.5m width was filled with two layers of angular broken stones.
- A 4cm thick gravel surfacing was laid at top and cross-drains were provided at spacing 90cm.

Souvik 2021 - Instead of kerbs a layer broken stones were used to impart lateral stability.

Note:- From Roman roads to Tudor roads two design considerations were common

- (i) subgrade soil was constructed on a level surface
- (ii) large foundation stones were used to make bottom most layer.
- (iii) The major change in designing of roads was introduced by Macadam.

(iv) MACADUM ROADS :-



- Macadam roads differs from previous design in following aspects.

- (a) soil subgrade was also laid at a cross-slope of 1 in 36 to avoid the seepage of water in it.
- (b) He was the first one to suggest that large foundation stones are not required to be placed at the bottom layer.
- (c) Similarly the next layer of pavement also was constructed above this layer with broken stones of smaller size.
- (d) Though the total thickness of construction was less, but load distribution was comparatively better.
- (e) The size of broken stones at top was decided on the basis of stability under animal drawn vehicle.

NOTE:- (i) Different types of specification were developed for the construction of bituminous pavement layers for use in base and surface courses.

(ii) Some of the specs used in India are as follows.

- (a) WBM (Water Bound macadam)
- (b) PM (Penetration macadam)
- (c) BM (Bituminous macadam)
- (d) DBM (Dense Bound macadam)
- (e) WMM (Wet mix Macadam)

DEVELOPMENT OF ROADS IN INDIA

Mohenjo - Daro → Ashoka → Mughal → Britishers.

(I) Government of India passed the resolution in 1927 for appointment of a committee to examine the situation and development of roads in India under the leadership of MR Jaykar.

- Major recommendations of this comt. are as follows

- (i) The road development in the country should be the subject of NATIONAL INTEREST
- (ii) An extra tax should be levied on petrol from the users for development of roads & would be termed as central road fund.
- (iii) A semi-official technical body should be formed to give technical know how for development of roads.
- (iv) A research organisation should be instituted to carry out research and develop new techniques of road development.

(II) As per the recommendation of Jaykar committee CRF (Central Road fund) was established in 1929

(III) At the approval of govt a semi-official technical body was formed in 1934 termed as IRC (Indian road congress)

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(IV) In 1939, motor vehicle act was brought in place to frame the laws and ordinances relating to traffic. [It governs driver vehicle and owner operation]

(V) A conference of chief Engineer of all states was called by IRC for collective development of roads in India in 1943 [1st 20 year plan] (1943-1963) [It was completed in 1961 only] termed as nagpur road conference,

In this target of 16 km / 100 km² area of country for development of roads was to be achieved.

(VI) In 1950 central road research institute (CRR) was established for carrying out research of road technology.

(VII) In 1956 national highway act was passed for following purposes.

- To declare certain selected highway as NH
- To enter into any land for carrying out survey.
- To acquire land & take possession for development of highway.

(IX) Due to the early completion of first 20 yr plan in 1961, second 20 yr plan was initiated in 1961 (1961-1981) in Bombay for development of 32 km / 100 km² of area.

(x) In 1973, HRB (Highway research board) of IRC was setup to give direction and guidance for research activities in India

(xi) In 1978, national transport policy committee (NTPC) was appointed to prepare a comprehensive national transport policy for the country for next 10 yrs (decade).

(xii) In 1981, third 20 yr road development plan was introduced. (1981-2001), Lucknow, 82 km / 100 km²

(xiii) In 1988, NHAI act was passed.

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2021 (xiv) In 2000 PM USY was launched by Indian govt to increase

(xv) Fourth 20 yr road development plan should have been introduced in 2001, but on the insistence of govt IRC

JAYKAR COMMITTEE (1927)

prepare d Road
development plan
vision 2021 &
2025

CENTRAL ROAD FUND (1929)

INDIAN ROAD CONGRESS (1934)

MOTOR VEHICLE ACT (1939)

NAGPUR ROAD CONFERENCE (1943)
16 km / 100 km², [1943 - 1963] (1961 → finished)

CENTRAL ROAD RESEARCH INSTITUTE (1950)
(CRR)

NATIONAL HIGHWAY ACT (1956) → Proposed

SECOND 20-YR ROAD PLAN (BOMBAY)
[1961 - 1981] → 32 km / 100 km²

HIGHWAY RESEARCH BOARD (1973)
- indicate CRR what to do

NATIONAL TRANSPORT POLICY COMMITTEE (1978)
- to connect all village road to NH.

THIRD ROAD 20 YR PLAN (1981-2001), 82 km / 100 km²

NHAI Act Passed (1988)

PMASy (Pradhan mantri gram sadak
Yojana) (2001)

ROAD DEVELOPMENT PLAN VISION 2021 & 2025

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COMPARISON B/W VARIOUS 20 YR ROAD DEVELOPMENT PLAN.

NAME	1 st 20 yr plan NAGPUR	2 nd 20 yr plan BOMBAY	3 rd 20 yr plan LUCKNOW
DURATION	1943 - 1963 (completed in 1961)	1961 - 1981	1981 - 2001
ROAD DENSITY	16 km/100 km ²	32 km/100 km ²	82 km/100 km ²
ROAD PATTERN	star & grid	—	—
EXPRESS HIGHWAY	—	1600 km	20000 km added
CLASSIFICATION OF ROADS	NH, SH, MDR ODR, VR	SAME	(I) PRIMARY ROADS EH, NH (II) SECONDARY SH, MDR (III) TERTIARY/RURAL VR, ODR

NH = National Highway
 SH = State Highway
 EH = Express Highway
 ODR = Other Dist. Road
 VR = Village Road

NOTE:- length of the road as per 3rd 20 yr road plan (km)

$$(i) \text{ Total length of road} = \max \left\{ \begin{array}{l} 4.74 \times \text{No. of towns and villages} \\ \text{Road density} \times \text{Area} \end{array} \right.$$

$$(ii) \text{ length of NH} = \frac{\text{Area (km}^2\text{)}}{50}$$

$$(iii) \text{ length of SH} = \max \left\{ \begin{array}{l} \frac{\text{Area (km}^2\text{)}}{25} \\ 62.5 \times \text{no. of towns} - \text{length of NH} \end{array} \right.$$

$$(iv) \text{ length of MDR} = \max \left\{ \begin{array}{l} \frac{\text{Area (km}^2\text{)}}{12.5} \\ 90 \times \text{no. of towns} \end{array} \right.$$

$$(v) \text{ length of ODR and VR} = \text{Total} - (ii) - (iii) - (iv).$$

Q) The area of a certain district in India is 13400 km^2 and there are 12 towns as per census. Determine the length of diff. categories of road to be provided in the dist. by the year 2011.

Sol Total length = max $\left\{ \begin{array}{l} 4.74 \times 12 = 56.88 \text{ km} \\ \frac{84}{100} \times 13400 = 10988 \text{ km} \end{array} \right.$

Total length of NH = $\frac{\text{Area (km}^2\text{)}}{50} = \frac{13400}{50} = 268 \text{ km}$

" " " SH = max $\left\{ \begin{array}{l} \frac{13400}{25} = 536 \text{ km} \\ 62.5 \times 12 - 268 = 482 \text{ km} \end{array} \right.$ = 536 km

MDR = max $\left\{ \begin{array}{l} \frac{13400}{12.5} = 1072 \text{ km} \\ 90 \times 12 = 1080 \text{ km} \end{array} \right.$ = 1080 km

ODR & VR = $10988 - 268 - 536 - 1080$
= 9104 km

∴ Total = 10988 km	primary = EH + NH = 0 + 268 = 268 km
NH = 268 km	
SH = 536 km	second = SH + MDR = 536 + 1080 = 1616 km
MDR = 1080 km	
ODR & VR = 9104 km	Rural = ODR + VR = 9104 km

HIGHWAY PLANNING :-

- Highway planning is of great importance when the funds available are limited, whereas the total requirement is much higher.
- The objective of highway planning are as follows
 - To plan the overall network of road efficiently in minimum cost
 - To divide the overall plan in phases to decide the priorities.
 - To plan for future requirement and improvement of the roads.

CLASSIFICATION OF ROADS :-

- Roads are classified as follows :-

(I) on the basis of duration of their use.

(a) All weather roads

- Paved and non paved roads.

(b) Fair weather roads

- Surface and non surface roads.

(II) on the basis of traffic volume

(III) on the basis of load transported / Tonnage

(IV) on the basis of location and function.

(a) National Highway → This are main highway running through length and width of country connecting major ports foreign highway, capitals of large states and large industrial area, tourist and places of strategic importance (Defence)

(b) State highway → are arterial roads of states connecting the national Highway of adjacent state, district head quarters and important cities.

(c) Major District Road → They are important roads within a district serving area of production, markets and connecting

(d) Other district Road → are roads serving rural areas of production and providing them outlet to the market.

(e) village road → are roads connecting villages

NOTE:- Urban roads are also classified as follows.

(a) Arterial roads.

(b) Sub-Arterial.

(c) collector street.

(d) local streets.

TYPES OF ROAD PATTERN

The various types of road pattern may be classified as follows.

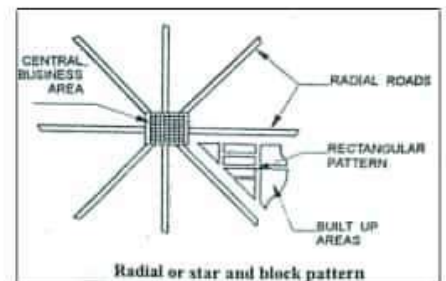
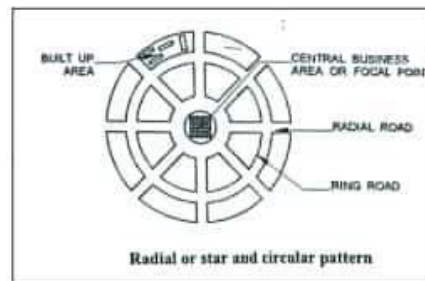
(a) RECTANGULAR / BLOCK pattern

(b) HEXAGONAL

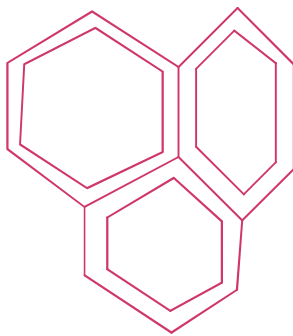
(c) RADIAL OR STAR & BLOCK

(d) RADIAL OR STAR & CIRCULAR

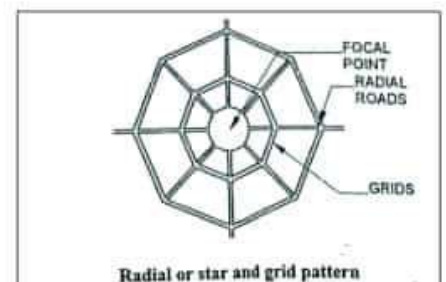
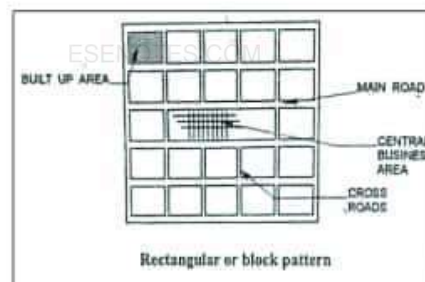
(e) RADIAL OR STAR & GRID



Rectangular / Block pattern



Hexagonal pattern.



ROAD PATTERN

spider web

MAXIMUM SATURATION SYSTEM / UTILITY FACTOR :-

- The system is used to choose, the best alignment amongst the various options available
- It depends upon population and production.

Rules to decide the factor

- Souvik 2021
- (i) Provide UF of 0.5 to the lowest population. and increase it by multiplying with 2 for next range of population.

Eg $\rightarrow 0.5 \quad 1 \quad 2 \quad 4 \quad 8 \quad 16 \dots$

- (ii) Provide UF of "1" to the agricultural production
(iii) Provide UF to industrial production as per the weightage.

22 DEC

- Q) There are four alternate proposal of a road plan for backward district, the details are given as follows. Justify with reason which proposal is the best assuming utility units of 0.5, 1, 2, 4 for the five population range and 1 & 5 per thousand ton of agricultural and industrial product served.

Proposal	Total Road length (km)	no. of towns & villages served with population range					Productivity (tonnes)	
		<2K	2K-5K	5K-10K	10K-20K	>20K	Agricultural	Industrial
P	500	100	150	40	20	3	150	20
Q	600	200	250	68	28	3	220	25
R	700	270	350	82	36	4	300	35
S	800	280	410	91	41	4	400	42

Sol Proposal P

$$\begin{aligned} \text{Total utility} &= 100 \times 0.5 + 150 \times 1 + 40 \times 2 + 20 \times 4 + 3 \times 8 + \frac{150 \times 1}{1000} + \frac{20 \times 5}{1000} \\ &= 384 + 0.15 + 0.1 \\ &= 384.25 \end{aligned}$$

$$\text{utility per km of road} = \frac{384.25}{500} = 0.7685$$

Proposal Q

$$\begin{aligned} \text{Total utility} &= 200 \times 0.5 + 250 \times 1 + 68 \times 2 + 28 \times 4 + 3 \times 8 + \frac{220}{1000} \times 1 + \frac{25}{1000} \times 5 \\ \text{UF} &= \frac{\text{Total ut}}{600} = 1.037 \end{aligned}$$

Proposal R

$$\begin{aligned} \text{Total utility} &= 270 \times 0.5 + 350 \times 1 + 82 \times 2 + 36 \times 4 + 4 \times 8 + \frac{300}{1000} \times 1 + \frac{35 \times 5}{1000} \\ \text{UF} &= \frac{\text{Total utility}}{700} = 1.18 \quad \checkmark \end{aligned}$$

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Proposal S

$$\text{Total utility} = 280 \times 0.5 + 410 \times 1 + 91 \times 2 + 91 \times 4 + 4 \times 8 + \frac{400 \times 1}{1000} + \frac{42 \times 5}{1000}$$

$$UF = \frac{T \cdot U}{800} = 1.1608$$

proposal $R > S > Q > P$

ENGINEERING SURVEY FOR HIGHWAY ALIGNMENT

- In order to decide or finalise the probable alignment following survey are being carried out.

(i) MAP STUDY :- various alignments are drawn on map passing through min number of obstruction and maximum utilisation area.

(ii) Reconnaissance : It is done by visiting the location under consideration to identify the features which are not available on map or which are changed over the period of time.

- Feasibility is also checked for the possible alignment in this stage

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(iii) Preliminary survey :- In this survey (chain, compass, levelling, soil investigation, drainage provision, traffic studies) are carried out and max utilisation area

(iv) Detailed survey :- In this planning designing, material estimation and cost estimation etc are done and a DPR is prepared.

HIGHWAY

- AD of a highway deals with all the dimension and layout of visible features of highway such that it provides maximum efficiency at min cost and reasonable safety.
- Geometric design of Highway deals with :-
 - (a) cross-sectional element.
 - (b) sight-distance consideration.
 - (c) horizontal alignment detail.
 - (d) vertical alignment detail.
 - (e) intersection detail.
- For designing of highway elements following factors are considered

(i) Design speed

- Design speeds of vehicle depends upon type of road over which it ply (moves) eg- NH, SH etc.
- Design speed of vehicle decide the cross-sectional elements; width, clearance, SD, radius of curve, super elevation, transition, gradient, length of summit.

(ii) Topography :-

- Topography or terrain condition influence the design speed, that in turn governs the designing the highway element as above.

For eg, in plain terrain on SH, $V_{perm} = 100 \text{ km/hr}$ whereas the same speed on rolling terrain, $V_{per} = 80 \text{ kmph}$ and on mountainous terrain, $V_{per} = 50 \text{ kmph}$.

(iii) Traffic factors :-

- Traffic factor in crudes vehicle characteristics and human characteristics.

- In order to design highway, std. size of vehicle is considered (car) in case of mixed traffic operation.

- Human traffic includes physical mental and psychological characteristics. or driver and pedestrian.

(iv) Design hourly volume and capacity :-

- The traffic vol. fluctuates over the period of time, hence the designing must be done for peak hours, but it will be highly uneconomical to do so, hence designing is being done for reasonable value of traffic volume termed as design hourly volume.

(v) Environmental factors :-

- Factors like aesthetics, air pollution, noise pollution, landscaping also governs the designing of highway elements.

HIGHWAY ELEMENTS :-

(I) HIGHWAY CROSS-SECTION ELEMENTS :-

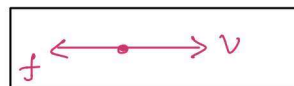
(a) Pavement surface characteristics :-

- It depends upon pavement type, which in turn depends on availability of material, cost, composition of traffic, climatic conditions, mtd of construction availability.
- Pavement surface characteristics include :-
 - (i) friction (ii) unevenness (iii) Reflection properties
 - (iv) Drainage of surface water.

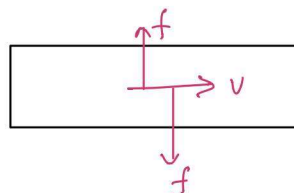
FRICTION / SKID RESISTANCE :-

- It decides the operating speed and min. distance req. for stopping the vehicle
- It is further classified into two

- longitudinal friction



- lateral / Transverse friction



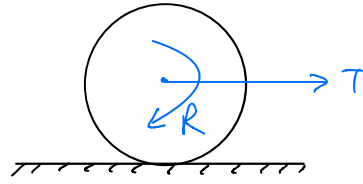
- Maximum friction is developed when brakes are applied upto complete extent.

- Friction also governs rotational, translational movement of the vehicle.

For uniform condition

$$\text{Rotation} = \text{Translation}$$

$$1(R) = 2\pi r(T)$$



	Rotation	Translation	Impact
- if long. friction is more	X	✓	Tyre burn
if long. friction is less	✓	X	NO movement of vehicle. (Fuel Burn)

NOTE:- (i) when the path travelled along the road surface is more than the circumferential movement of wheels due to their rotation, it is termed as "SKID".

$$T_M > R_M$$

For pure skid $R_M = 0$, $T_M \neq 0$.

Eg:- Brake applied, rotation stops but skidding occurs.

(ii) when the circumferential movement of wheel is more than the path travelled along the road surface it is termed as "SLIP".

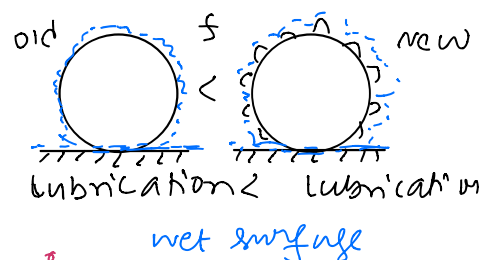
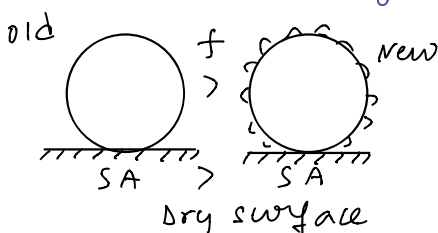
$$R_M > T_M$$

For pure slip $T_M = 0$, $R_M \neq 0$.

Eg:- when wheels of vehicle in mud. It rotates but donot move ahead.

- Friction depends upon following factor.

- Type of pavement surface (Bitumen, concrete, earthen etc)
- Roughness of pavement (texture)
- Condition (Dry or wet, rough or smooth)
- Condition of tyre. (New, old)
- Speed of vehicle. (High speed less friction, low speed more friction)
- Extent of Brake application. (full, partial)
- Load and tyre pressure
- Temperature of tyre and pavement. ($\text{Temp} \uparrow$, $\text{PR} \uparrow$, $f \uparrow$)



NOTE:- New tyre is more dependable in adverse condition.

- For the calculation of stopping distance, the longitudinal friction coeff. is taken to be (0.35-0.40)

at $f=0.4$, the Retardation available is approx $4m/s^2$

- And transverse friction is taken to be 0.15

UNEVENNESS :-

- Presence of undulations on the pavement surface is called unevenness

- This unevenness results in

- increase fatigue.
- Reduces speed.
- Increases fuel consumption.
- Increases wear and tear of vehicle.
- Increase chances of accidents.

- The unevenness of pavement is measured with the help of equipment BUMP INDICATOR (BI) in terms of "unevenness index" which is the vertical undulations of pavement surface recorded per unit length of road. (In India mm/km is used)

26 Dec - The different values of unevenness index and the corresponding serviceability of road is as follows.

UNEVENNESS INDEX (mm/km)	Type of road
<1500	Good
1500-2500	satisfactory
2500-3500	BAD
>3500	uncomfortable

NOTE:- Internationally the riding quality of pavement surface is quantified in terms of roughness and is expressed as "INTERNATIONAL ROUGHNESS INDEX"

2) It can also be related with unevenness index as follows.

$$BI = 630 (IRI)^{1.12}$$

(mm/km)
(m/km)

Unevenness of the pavement depends on following factors

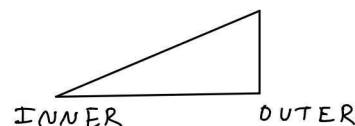
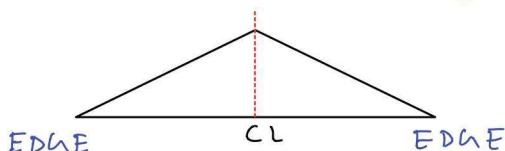
- (i) Improper compaction.
- (ii) Use of improper construction methods.
- (iii) Use of inferior quality materials.
- (iv) Improper surface and sub-surface drainage.
- (v) Poor maintenance of pavement surface.

REFLECTIVE PROPERTIES :-

- visibility over the pavement surface depends upon its colour and light characteristics, the glare caused by the reflection of head lights is high on wet pavement surface than dry pavement.
- light coloured pavement gives good visibility at night but produces more glare during sunlight, on the other hand dark colour pavement offers good visibility at day and poor at night.

(b) CROSS SLOPE / CAMBER :-

- It is the slope provided to the road surface in the transverse direction to drain the rain water to the road surface to avoid the following :-
 - (a) stripping of bitumen from the aggregate in the presence of water.
 - (b) Swelling and heaving of subgrade in case water seeps up to it.
 - (c) To avoid the slipping of vehicle over the wet pavement
 - (d) To avoid the glare in wet pavement.
- On straight roads it is provided by raising the centre of the carriageway w.r.t the edges forming the crown on the highest point along the centre line
- On horizontal curve with super-elevation, the drainage is provided by lifting the outer edge

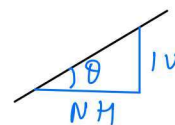


- It is represented by any of the following ways.

(a) as % but eg. cross-slope 5% $\tan \theta = 5/100$.

(b) as fraction, eg. $CS = 1 \text{ in } 20$ $\tan \theta = 1/20$
 $\text{or } 1/20 \text{ or } 0.05$

1 in 20 means, 1V : 20H



- The value of cross-slope depends upon following factors.

(i) Type of pavement surface.

(ii) Amount of rainfall.

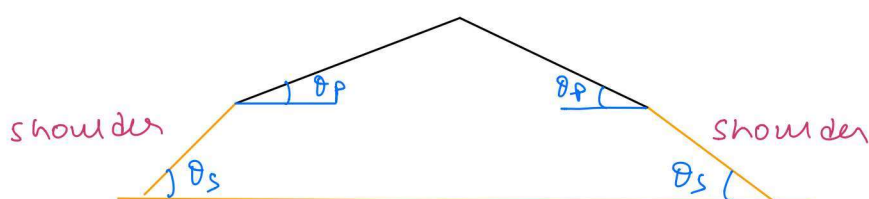
cross-slope provided would be comparatively increased if amt. of rainfall is more and pavement surface is permeable.

TYPE OF ROAD SURFACE	RANGE OF CROSS-SLOPE	
	HEAVY RAINFALL	LIGHT RAINFALL
- cement + concrete & high type bituminous surface	2%	1.7%
- Thin bituminous surface	2.5%	2%
- WBM and gravel pavement	3%	2.5%
- Earthen road	4%	3%

- The cross-slope of shoulder should be more than that of pavement, so as to avoid accumulation of water at junction of two.

- Cross slope of shoulders should be 0.5% more than cross slope of adjoining pavement, having min value of 3% & max value of 5%.

$$5\% \nless CS_{\text{shoulder}} = (0.5\% + CS_{\text{pavement}}) \nless 3\%$$



- The CS on express ways for carriage way and paved shoulder and edge strip with bituminous surface is 2.5% with rainfall exceeding 1000 mm and 2% for places having rainfall less than 1000 mm.

- If too steep CS is provided

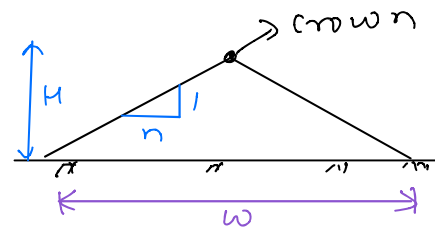
- (i) Toppling of slow moving & over-loaded vehicles.
- (ii) Tendency of most vehicles to travel along the centre line
- (iii) uncomfortable side thrust and drag on the vehicle.

- cross-slopes are of following types

(i) Straight line camber :-

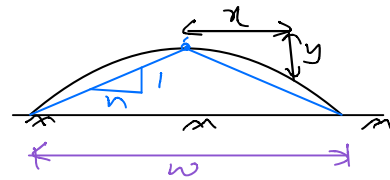
$$\frac{1}{n} = \frac{H}{W/2}$$

$$H = \frac{W}{2n}$$



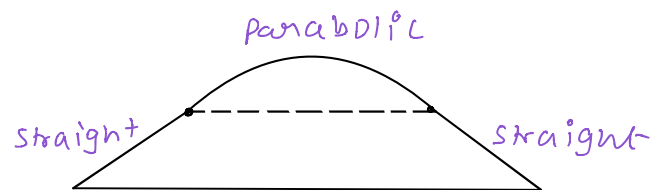
(ii) Parabolic camber :-

$$y = \frac{2x^2}{W}$$



(iii) Composite camber :-

parabolic at top.
straight at side.



NOTE:- For cement concrete pavement straight line camber is preferred as it is easier to lay.

Q) In the region where the rainfall is heavy, two types of pavement are to be constructed.

- (a) Two lane SH with bituminous concrete surface
 - (b) MDR of WBM pavement 3.8m wide.
- What should be the height of the crown, with respect to the edges in this 2 cases, assuming st. line camber and width of two lane SH as 7m.

Sol (a) $\tan \theta = \frac{2}{100} = \frac{1}{50}$ $n = 50$

$$\frac{H}{(w/2)} = \frac{1}{N}$$

$$H = \frac{700}{2 \times 50} = 7 \text{ m} = 70 \text{ mm}$$

(b) $H = \frac{w}{2N} = \frac{3800}{2 \times 33.3} = 57 \text{ mm}$
 $\downarrow$
 $3\% = \frac{3}{100} = \frac{1}{33.3}$

(c) WIDTH OF PAVEMENT / CARRIAGEWAY :-

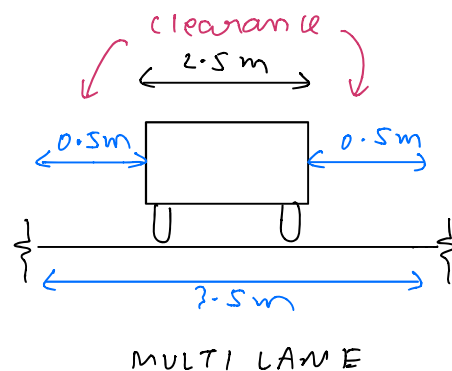
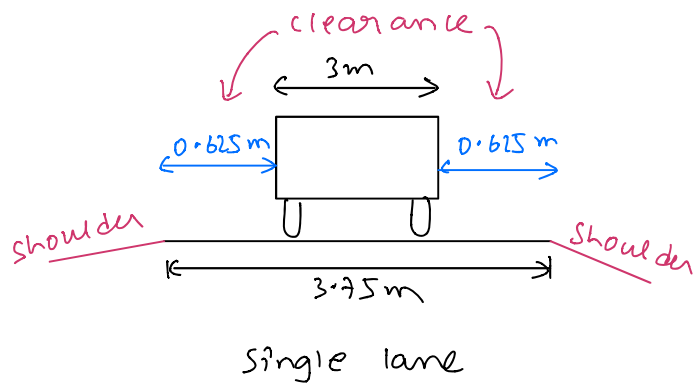
- width of pavement depends upon width of traffic lane and number of lanes, the portion of carriageway width i.e. intended for one line of traffic movement is called as traffic lane.
- width of traffic lane is decided on the basis of type of vehicle moving on it along with some clearance in both the side.
- Passenger car is considered as std vehicle to decide the width of carriageway.

(width of passenger car = $2.44 \approx 2.5 \text{ m}$)

- For rural highway, if pavement has two or more lanes (multilanes), width of single lane = 3.5 m
- The no. of lanes to be provided depends upon traffic volume.
- The width of the carriageway for different conditions are as follows:-

Type of road	width of carriageway.
1. SINGLE LANE ROAD	3.75
2. TWO LANE without raised KERBS	7
3. TWO LANE with raised KERBS	7.5
4. Intermediate carriage way	5.5
5. MULTILANE PAVEMENT	3.5m per lane

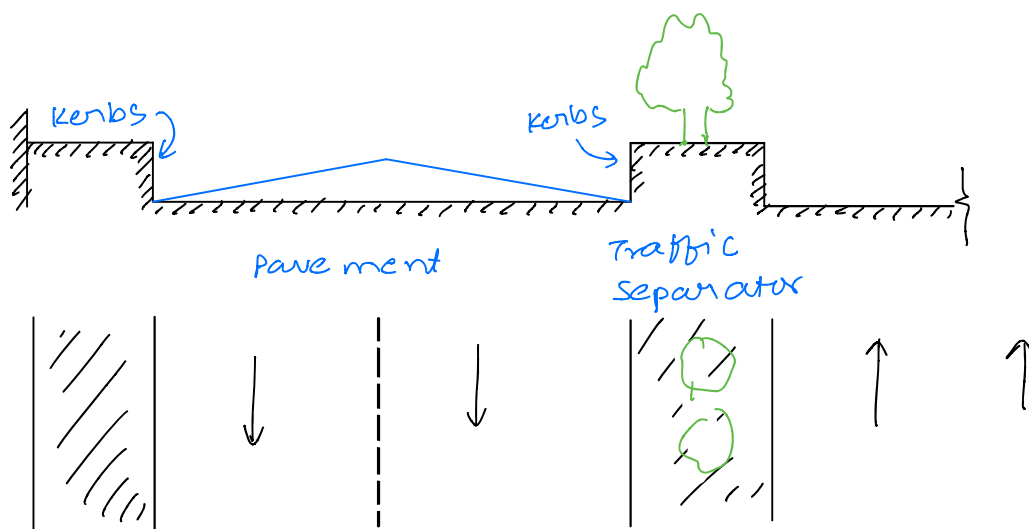
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(d) MEDIANS / TRAFFIC SEPARATOR :-

- In highways with divided carriageway or pavement a median is provided b/w two sets of traffic lanes intended to divide the traffic moving in opposite directions.
- The main function of median is to prevent head on collision between vehicles moving in opposite direction.
- It also serves following other functions.
 - (a) To channelize traffic into streams.
 - (b) To protect the pedestrians.
 - (c) can be used to reduce the glare of the head light of opposite moving vehicle by providing green cover on it.
 - (d) To segregate the slow traffic.
- Medians can be provided in following forms :-
 - (a) As pavement marking.
 - (b) Physical divider (Mechanical separator).



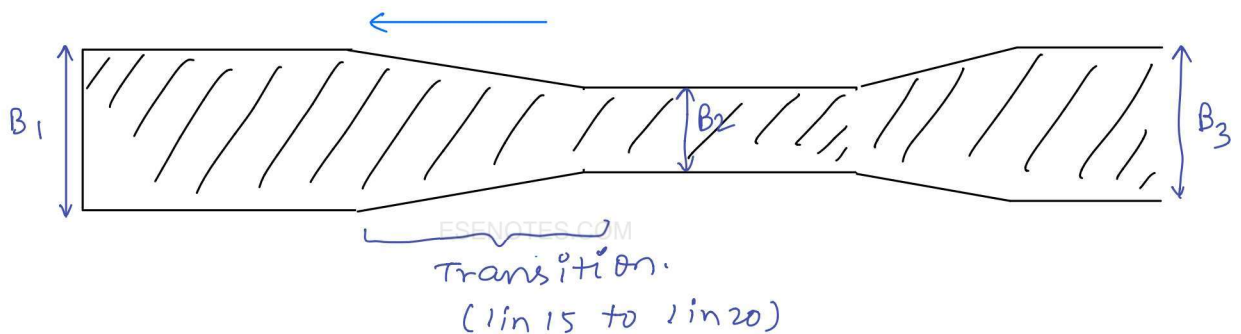
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- Desirable width of median is (8-14) m

It includes the provision for future expansion of roads.

In order to reduce the glare of headlight of opposite vehicle a min of 6m width is required

- IRC recommends min width of 5m for median, that can be reduced to 3m where land is restricted
- On bridges width required for median is in the range of (1.2 - 1.5) m.
- The median should be normally of uniform width throughout the length of the pavement or carriage way, but where its width is changed a transition of 1 in 15 to 1 in 20 must be provided.



- In urban areas min. width of median to be provided is 1.2m & desirable min is 1.5m.
- For expressways min. width to be provided is 12m & desirable is 15.5m

(e) KERB :-

- Kerb indicates the boundary between the pavement & median or foot path or island or shoulder.
- Kerbs are mainly divided in following three categories.

(a) LOW KERB :- It is of height 100mm (10cm) above the pavement surface (edge) with a slope to enable the vehicle to climb the kerb at slow speed.

- It is also termed as "MOUNTABLE KERB" which encourages the traffic to remain in the through traffic lanes yet allows the driver to enter the shoulder

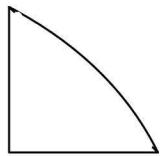
area at slow speed.

(b) SEMI-BARRIER TYPE KERB :- It is provided on the periphery of the road way where the pedestrian traffic is high. This type of kerb is of height of about 150 mm above the pavement edge with slope of 1:1 on top 75 mm.
(batter)

- This kerb prevents encroachment by parking vehicles. but in case of emergency it is possible to drive over this kerb with difficulty.

(c) Barrier type curb :- It is provided in built up areas adjacent to foot paths with considerable pedestrians traffic.

- height is 200 mm (200 mm) above the pavement edge of steep slope of 1V:0.25H



low kerb

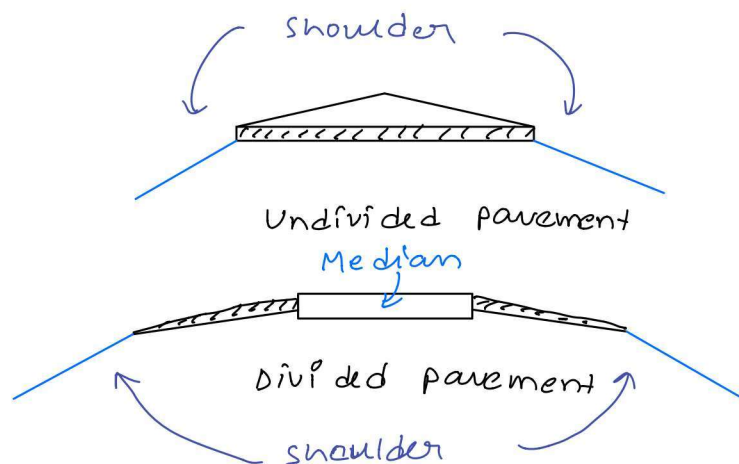


Semi-barrier.



Barrier type

(f) SHOULDER :-



- shoulders are provided on both sides of the pavement all along the road in the case of undivided carriageway and on outer edge of divided carriageway
- The earthen shoulder should have sufficient strength to carry the vehicular load in case of emergency.

- The minimum shoulder width as per IRC is 2.5m

- Shoulder serves following functions :-

- (a) It imparts structural stability to the pavement.
- (b) It increases the capacity and operating speed of pavement.
- (c) In emergency it can be used as mode of movement.
- (d) It also act as a service lane for the vehicles that are disabled.
- (e) The surface of the shoulder may be rougher than the traffic lanes so as to discourage the vehicle to fly over it.
- (f) The colour of the shoulder should be different from that of pavement, so as to distinct between the two.

(9) ROAD MARGIN :-

- The various elements included in the road margins are guard rails, foot path, drive way, cycle track, parking lane, lay-bay, front edge, roads and embankment slope.

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DEC 29 (a) Guard Rails - These are provided at the edge of the shoulder when the road is constructed on a fill so that vehicles are prevented from running off the embankment, especially when the height of fill is more than 3m

(b) Foot path / Side walk :- In order to provide safe facility to the pedestrian to walk along the roadway, footpath / side walk are provided in urban areas, where the pedestrian traffic is comparatively more.

- Its min. width required is 1.5m and desirable min is 2m

- It is provided with cross slope of 2.5-3% and is comparatively smoother surface than pavement.

(c) Drive ways :- It connects the highway with local commercial establishment like service station, bus station, restaurant etc.

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(d) Cycle track :- These are provided in urban areas for the safe movement of cycle traffic.

- The minimum width required is 2m for cycle track and it can be increased by 1m for each additional cycle way.

(e) Parking lanes :- These are provided on urban roads to allow kerb parking

- As far as possible only parallel parking should be allowed as it is safer for moving vehicles and space required would also be less.
- For parallel parking the min. width is 3m → Pushing back -

(f) Bus bays :- It may be provided by recessing the kerb to avoid the conflict with the moving traffic and must be located at least 7.5m away from intersection.

- It is used for safe loading & unloading of passengers.

(g) Lay Bys :- These are provided near public convenience to avoid the conflict with running traffic.

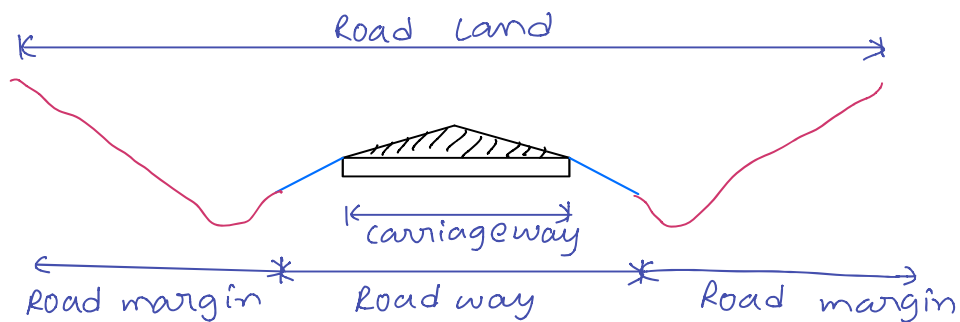
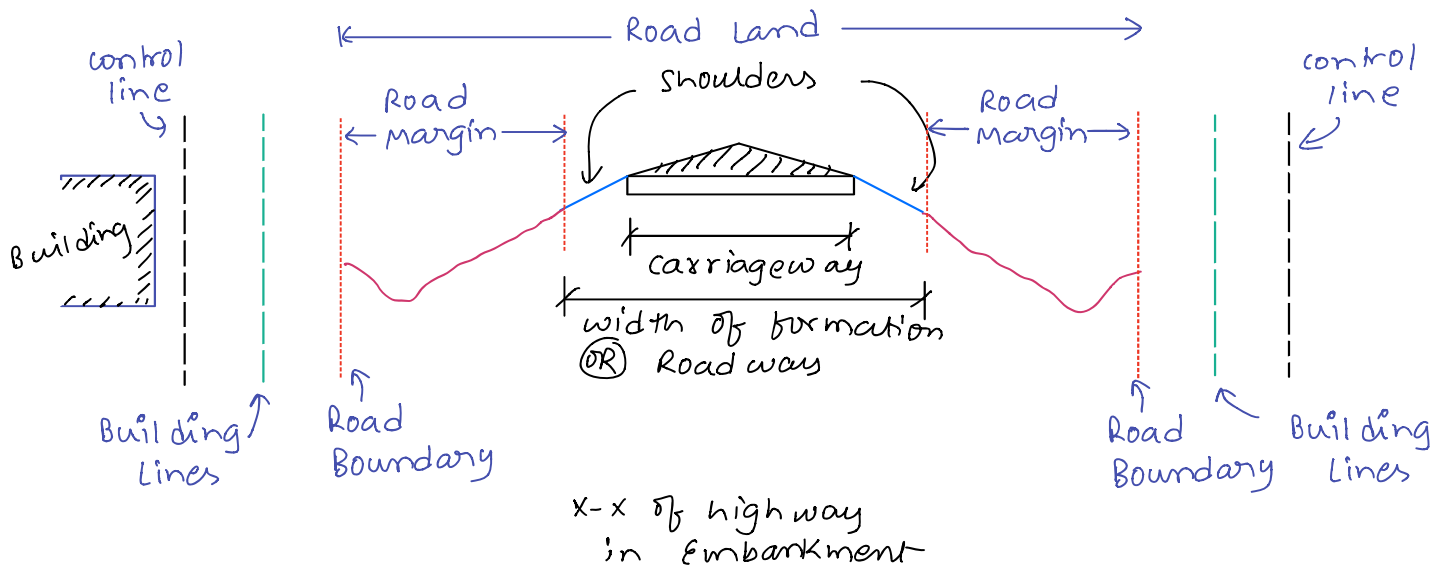
- Min width is 3m.

(h) Frontage Roads :- These are provided to give access to the property along the highway to control the access to the expressway.

(i) Embankment slope :- If pavement is constructed over embankment, slopes are also provided along it of magnitude 1 in 3.

NOTE: Roadway/width of formation :- It is the sum of width of pavement or carriage way including separator and the shoulder.

- Roadway width is the top width of highway embankment or bottom width of highway excluding the side ways cutting



- width of roadway of various class of roads are as follows.

Type of roads	Roadway width (m)	
	plain & Rolling	Mountainous / steep
(i) NH & SH		
(a) single lane	12m	6.25m
(b) two lane	12m	8.8m
(ii) MDR		
(a) single lane	9m	4.75m
(b) Two lane	9m	-
(iii) ODR		
(a) single lane	7.5m	4.75
(b) Two lane	9m	-
(iv) village Roads.	7.5m	4m

(ii) Road land / land width / Right of way :- It is the are of the land acquired for the road, along its alignment keeping in view its future expansion also.

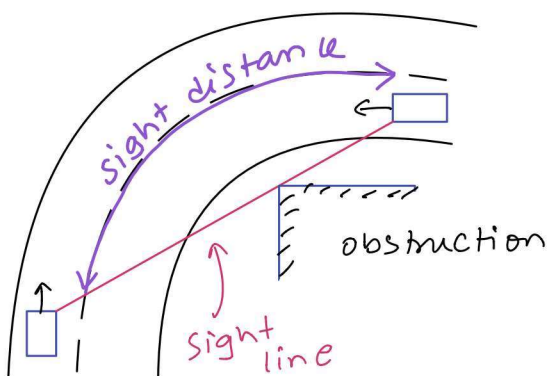
- construction of a particular type of building is only permitted with sufficient setback from the road boundary upto control line.

width of land for different roads in rural areas are as follows

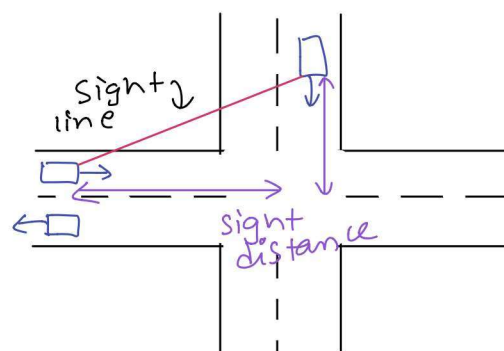
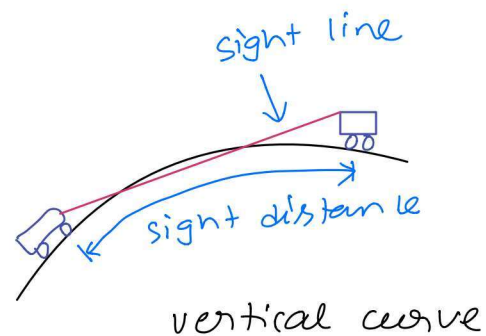
Road classification	PLAIN & ROLLING		MOUNTAENOUS	
	OPEN	BUILT AREA	OPEN AR.	BUILT
(i) Express way	90m	-	60 m	-
(ii) NH & SH	45m	30 m	24 m	20 m
(iii) MDR	25m	20 m	18 m	15 m
(iv) DDR	15m	15 m	15 m	12 m
(v) UR	12m	10 m	9 m	9 m

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(II) SIGHT DISTANCE :-



horizontal curve



At intersection

- Sight distance is the length of the road visible ahead to the driver at any instance. It is considered by taking the height of the eye of driver to be 1.2m and ht. of object is 0.15m

- Restriction to the visibility or SD will be caused in following case

(a) At horizontal curve:- when line of sight is obstructed by object at the inner side of the curve

(b) At a vertical curve, the LOS is obstructed by the road surface of the curve.

(c) At an uncontrolled intersection when a driver from one of the approach road is able to sight a vehicle from another approach roads towards it at the

when obstruction is not there, more SD is available

- The sight distance available to the driver is dependent upon following factors:-

(a) Features of Road. (e.g. horizontal curve, traffic conditions, position of obstruction etc)

(b) Height of driver's eye above road surface.

(c) Height of object " " " "

- Sight distance are of following types:-

(a) stopping sight distance.

(b) safe overtaking distance.

(c) safe sight distance for entering into uncontrolled intersection.

(d) Intermediate SD.

(e) head light SD.

(i) STOPPING SIGHT DISTANCE :-

- The min. distance visible to a driver ahead to safely stop a vehicle travelling at design speed without collision with any other obstruction is termed as stopping sight distance (SSD)
- It is also termed as "absolute min. sight distance" or "non-passing sight distance".
- Factors governing stopping sight distance.

(a) Total Reaction time of driver :-

- It is the time taken from the instant the object is visible to the driver to the instant brakes are applied effectively.
- This total reaction time is further divided into different components and can be explained as follows

(a) Perception time (P): It is the time req for the sensation received by the eyes or ears of the driver to be transmitted by the brain through the nervous system and spinal cord.

(b) Intuition time (I): It is the time req by the driver to understand the situation. It is also the time req for comparing different thoughts, regrouping and registering new sensation.

(c) Emotion time (E):- It is the time elapsed during emotional sensations and other mental disturbances, fear, anger, superstition etc, with reference to the situation to come into play.

- This time varies from person to person.
- And even for same person in diff. situation.

(d) Volition time (V): It is the time taken by the driver for final action, like application of brake.

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The above can be termed as "PIEV" analysis for total reaction time.

$$t_R = (P + I + E + V) \text{ time}$$

$t_{R, \min}$ = Reflex action time.

NOTE:-1) It is also possible that the driver may apply the brake or take any other avoiding action, like turning, by the reflex action, without normal thinking process, that is observed to be min time for avoiding the collision

2) This reaction time depends upon several factors like driver skill, type of obstruction involved, environmental condition, age, mental health.

(b) DESIGN SPEED :-

- During the total reaction time of the driver, the distance moved by the vehicle depends upon design speed.
- Braking distance i.e. dist. travelled by vehicle before coming to stop/halt also depends on design speed.

(c) Braking efficiency :-

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- It is defined as the % of force developed by application of brakes in relation to $f_{\max}$ friction available b/w tyre and road.

$$\eta_B = \frac{f}{f_{\max}} \times 100 = \frac{\text{force developed}}{\text{max. force available}}$$

If $\eta_B = 100\%$, skid will occur on application of brake, which is difficult to control.

hence braking force should not exceed, the friction force b/w wheels and tyres.

(d) Frictional resistance b/w Road & Tyre :-

- The frictional resistance developed b/w the road and tyre depends upon coeff of friction, higher is coeff. of friction, higher is the frictional resistance and lower is SSD

Souvik 2021 - coeff. of friction further depends upon condition of tyre & roads.

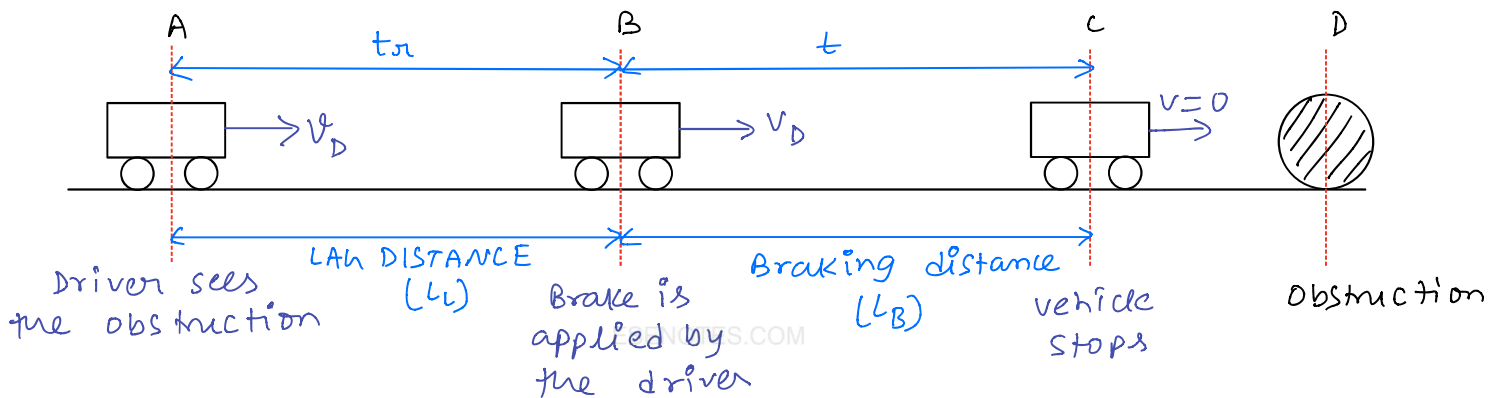
- It is further linked with the design speed of vehicle as follows.

Speed (kmph)	20-30	40	50	60	70	80	≥100
Longitudinal friction	0.4	0.38	0.37	0.36	0.36	0.35	0.35

(e) GRADIENT :-

- Gradient of the road add in either in distance or resistance of the vehicle depending upon type of gradient (up or down), thereby either increases or decreases SSD.

Analysis of stopping distance :-



$$SSD = L_L + L_B$$

- The stopping distance of vehicle include

(i) The dist. travelled by the vehicle at uniform design speed during the total reaction time termed as lag distance (L_L)

(ii) Dist. travelled by the vehicle after the application of brake until it stops termed as braking distance (L_B)

lag distance $L_L = v_D \times t_r$ (m) $v_D = \text{Design speed (m/s)}$
 $t_r = \text{reaction time (s)}$

OR

$$L_L = \frac{v \times 10^3}{60 \times 60} \cdot t_r \text{ (m)} \quad v \text{ (km/hr)} \quad [t_r = 2-4 \text{ sec}]$$

$$L_L = \frac{1000}{3600} v t \text{ (m)} \quad v \text{ (km/hr)}$$

$$L_L = \frac{5}{18} v_D t_r$$

$$L_L = 0.278 v_D t_r \text{ (m)}$$

As per IRC, $t_{rr} = 2.5 \text{ sec}$ for SSD.

Braking distance,

(i) For flat road:-

$\Delta KE = \text{WD by the friction}$

$$\frac{1}{2} m v_f^2 - \frac{1}{2} m v_i^2 = (-) F \cdot L_B$$

$$\frac{1}{2} m v^2 = f \cdot N \cdot L_B$$

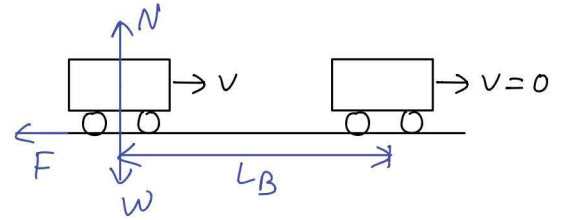
$$\frac{1}{2} m v^2 = f w L_B$$

$$v^2 = 2 g L_B f$$

$$L_B = \frac{v^2}{2 g f} \text{ (m)}$$

$$v = \text{m/s}$$

$$g = \text{m/s}^2$$



$$L_B = \frac{1}{254 \cdot 27} \frac{v^2}{f}$$

$$L_B = \frac{v^2}{254 f} \text{ (m)}$$

(K m/hr)

(ii) For gradient (Ascending & descending)

$\Delta KE = \text{WD}_{\text{Total}}$

$$0 - \frac{1}{2} m v^2 = -F \cdot L_B \mp w \sin \theta \cdot L_B$$

$$-\frac{1}{2} m v^2 = -f m g \cos \theta L_B \mp m g \sin \theta L_B$$

When $\theta = \text{small}$, $\theta \approx \sin \theta \approx \tan \theta = S$, $\cos \theta \approx 1$.

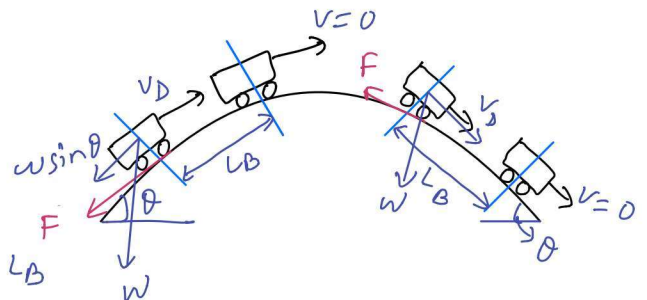
$$-\frac{1}{2} v^2 = -f g L_B \mp g S L_B$$

$$g L_B (f \pm S) = \frac{v^2}{2}$$

$$L_B = \frac{v^2}{2 g (f \pm S)}$$

$v \text{ (m/s)}$
 $g \text{ (m/s}^2\text{)}$

$$L_B = \frac{v^2}{254 (f \pm S)}$$



$+S = \text{Ascending gradient}$
 $-S = \text{descending gradient}$

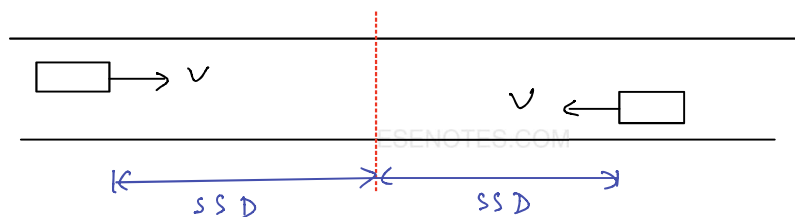
$$\therefore SSD = L_L + L_B$$

$$SSD = 0.278 V_D t_r + \frac{V_D^2}{254 (f \pm S)}$$

Dec 31) Note:- (i) SSD must be available at each and every section of the road.

(ii) As the SSD required on descending gradient is higher it is necessary to determine the critical value of SSD for the descending gradient on the roads with gradient & for two way traffic flow.

(iii) on the restricted width or on single lane road when two way movement of traffic is permitted the min stopping dist. is equal to twice the SSD to enable both the vehicles coming from opp side to stop.



(iv) If SSD cannot be provided on any stretch of road due to unavoidable reason, for the design speed available, the speed should be restricted either by installing speed limit regulation signs or by forced reduction of speed (by speed breaker)

- This is however temporary and efforts must be made to provide SSD over the period of time by changing the alignment of by moving the obstruction.

- IRC recommended the SSD values are as follows

Design speed (kmph)	20	25	30	40	50	60	65	80	100
SSD (m)	20	25	30	45	60	80	90	120	180

- 8) calculate the minimum SD required to avoid a head on collision of two cars approaching from the opp. direction at a 90 & 60 km/h. Assume a reaction time of 2.5s, $f = 0.7$ & brake eff of 50% in both the cases.

$$\begin{aligned} \text{sol} \quad \text{SSD}_1 &= 0.278 V_D t_{0.1} + \frac{V_D^2}{254(f \pm S)} \\ &= 0.278 \times 90 \times 2.5 + \frac{90^2}{254 \times 0.7 \times 0.5} \end{aligned} \quad \left| \quad \begin{aligned} \text{SSD}_2 &= 0.278 \times 60 \times 2.5 + \frac{60^2}{254 \times 0.7 \times 0.5} \end{aligned} \right.$$

$$\text{SSD}_1 = 153.66 \text{ m}$$

$$\text{SSD}_2 = 82.20 \text{ m}$$

$$\text{SSD} = 235.85 \text{ m}$$

- 9) Driver of a vehicle travelling at a speed of 80 kmph on a upgradient requires 9m len to stop after applying the brakes, than a driver travelling at same speed down the same grad. Find out the gradient of the road. ($f = 0.4$)

$$\text{sol} \quad V_D = 80 \text{ kmph}$$

down grad

up gradient

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$$(\text{SSD})_D = 80 \times 0.278 \times 2.5 + \frac{80^2}{254(0.4 - S)}$$

$$(\text{SSD})_{up} = 80 \times 0.278 \times 2.5 + \frac{80^2}{254(0.4 + S)}$$

$$(\text{SSD})_u = (\text{SSD})_d - 9$$

$$\frac{80^2}{254} \left(\frac{1}{0.4 + S} - \frac{1}{0.4 - S} \right) = 9$$

$$\frac{0.4 - S - 0.4 - S}{0.16 - S^2} = 0.357$$

$$-2S = 0.05712 - 0.357 S^2$$

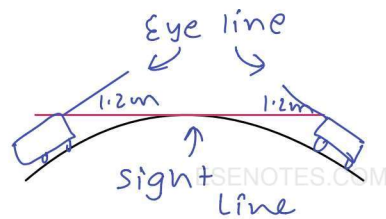
$$0.357 S^2 - 2S - 0.05712 = 0$$

$$S = 0.0284$$

$$S = 2.14\%$$

(II) OVERTAKING SIGHT DISTANCE :-

- If all the vehicles ply on a road at the design speed then there would be no need for overtaking, but practically this never happens.
- Hence vehicle moving at design speed needs to overtake slow moving vehicle.
- The min distance open to the vision of the driver of a vehicle intending to overtake slow vehicle with safety against traffic in opposite direction is termed as **OSD or safe passing distance / safe passing sight distance**.
- OSD is measured along c-line of the road considering **eye level of the driver at 1.2m above the surface, observing the object at height of 1.2m**.



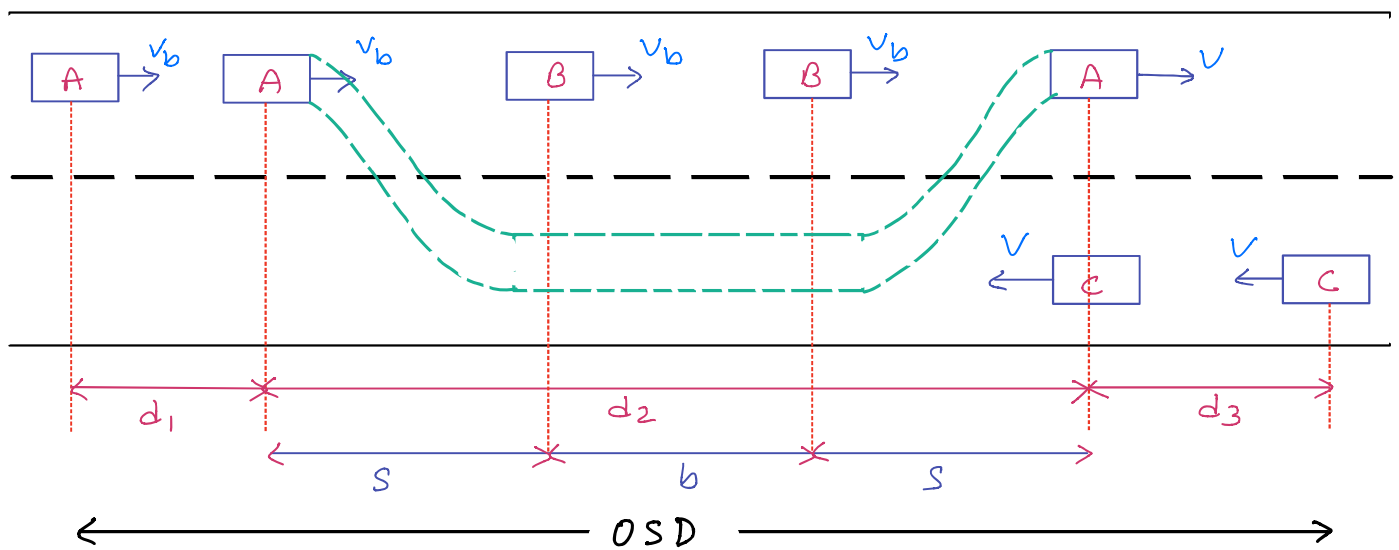
Factors affecting OSD :-

- speed of vehicle.
 - overtaking vehicle
 - over taken vehicle
 - vehicle coming from opp. direction.
- Reaction time (t_r). skill of the driver.
- Gradient of road.
- condition of overtaking vehicle which governs the rate of acceleration.
- Distance b/w two vehicles (overtaking & over taken)

Analysis of OSD :-

A = overtaking vehicle
B = overtaken vehicle

V = design speed

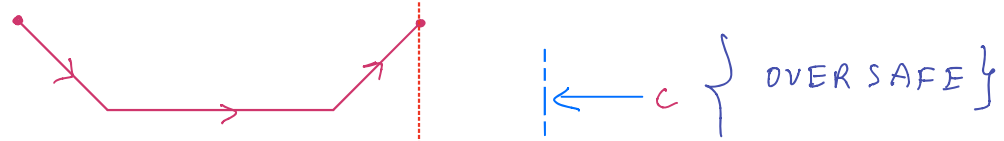
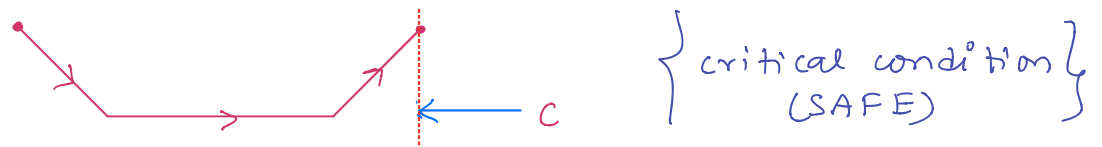


- The overtaking movement may be split up into three operations as follows.

(a) d_1 : It is the distance travelled by overtaking vehicle "A" during the reaction time " t_{or} " of the driver, before starting to overtake vehicle B at speed of V_b .

(b) d_2 : It is the distance travelled by vehicle "A" during the actual overtaking operation of time " T ".

(c) d_3 : It is the distance travelled by on coming vehicle "C", during the actual overtaking operation of "A" for time " T ".



- Assumption in analysis of OSD.

- (i) overtaking vehicle is forced to reduce its speed upto the speed of overtaken vehicle for reaction time.
- (ii) when the driver of vehicle find sufficient clear gap, decided within a reaction time to overtake.
- (iii) The vehicle A accelerates & overtakes the vehicle B within a distance " d_2 " during the time " T ".
- (iv) The distance d_2 can be splitted into three components.
- (v) During this overtaking time the vehicle "C" coming from opp. direction travels through distance d_3 .

$$d_1 = v_b t_{r1} = \overset{\text{km/h}}{\underset{\text{cs}}{0.278 v_b t_{r1}}}$$

$$t_{r1} = \text{Reaction time} = 2 \text{ sec.}$$

$$d_2 = 2S + b$$

$\rightarrow \text{time} = 0.7S$

$$S = (0.7 v_b + b) \text{ m}$$

$S = \text{Space headway (min. spacing b/w two vehicles which depends upon speed of the vehicle.)}$

$$S = (0.7 \times 0.278 v_b + b)$$

$$\boxed{S = (0.2 v_b + 6)} \quad v_b \text{ (km/h)}$$

$$\boxed{b = v_b T = 0.278 v_b T} \quad v_b \text{ (km/h)}$$

We know, $\left[D = ut + \frac{1}{2} at^2 \right]$

$$d_2 = v_b T + \frac{1}{2} a T^2$$

$$\boxed{d_2 = 0.278 v_b T + \frac{1}{2} a T^2}$$

$$2S + b = 0.278 v_b T + \frac{1}{2} a T^2$$

$a = \text{acceleration of overtaking vehicle (m/s}^2\text{)}$

$$2S + 0.278 v_b T = 0.278 v_b T + \frac{1}{2} a T^2$$

$$\boxed{T = \sqrt{\frac{4S}{a}}}$$

NOTE:- overtaking time " T " depends upon its avg acc'n and speed of overtaken vehicle. (S depend of v_b)

$$d_3 = V_d \times T = 0.278 V_d T \quad V_d = \text{design speed (kmph)}$$

$$\therefore OSD = d_1 + d_2 + d_3$$

$$OSD = (0.278 V_b t_m) + (0.278 V_b T + \frac{1}{2} a T^2) + 0.278 V_d T$$

NOTE:- In case the speed of overtaken veh. is not given it may be taken as. 4.5 m/s or 16 kmph less than design speed

$$\text{or } V_b = V_d (\text{m/s}) - 4.5 \text{ m/s}$$

$$V_b = V_d (\text{kmph}) - 16 (\text{km/h})$$

Jan-1 - The acceleration of the overtaking vehicle varies depending upon the several factors

- Type of vehicle.
- Condition of vehicle.
- Load on the vehicle.
- Speed of vehicle.
- characteristics of driver.

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- Max^m overtaking acc'n at different speed are as follows.

Speed (kmph)	Acceleration (m/s ²)
25	1.41
30	1.30
40	1.24
50	1.11
65	0.92
80	0.72
100	0.53

Effect of gradient on OSD :-

- The overtaking sight distance requirement may increase on ascending as well as descending gradient

- In descending gradient acc'n of overtaking vehicle inc, but overtaken vehicle may also acc. to a greater distance "b" during the overtaking time. Thus we cannot comment as there are two variables

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- In ascending gradient, the act of overtaking vehicle will be less but overtaken vehicle would also move with lower velocity, hence we cannot comment on OSD.
- Hence on mild gradient (as of that in plain & rolling terrains) the OSD at both ascending & descending gradient are generally equal to that of level stretch.
- However for steeper grades the OSD should be greater than min. OSD at plane level.
- OSD on two lane highway for different speeds are as follows:-

Speed (kmph)	TIME component - (s)			OSD (m)
	overtaking operation	opposing vehicle	Total	
40	9	6	15	165
50	10	7	17	235
60	11	7.2	18	300
65	11.5	7.5	19	340
80	12.5	8.5	21	470
100	14	9	23	640

NOTE:-> Here overtaking time may be increased by about $\frac{2}{3}$ rd to take in to consideration the distance covered by the vehicle from the opposite direction in case of two way traffic road during overtaking operations.

2) It is desirable that OSD should must be available on most of the road stretches.

- On road stretches with two way traffic.

$$OSD = d_1 + d_2 + d_3$$

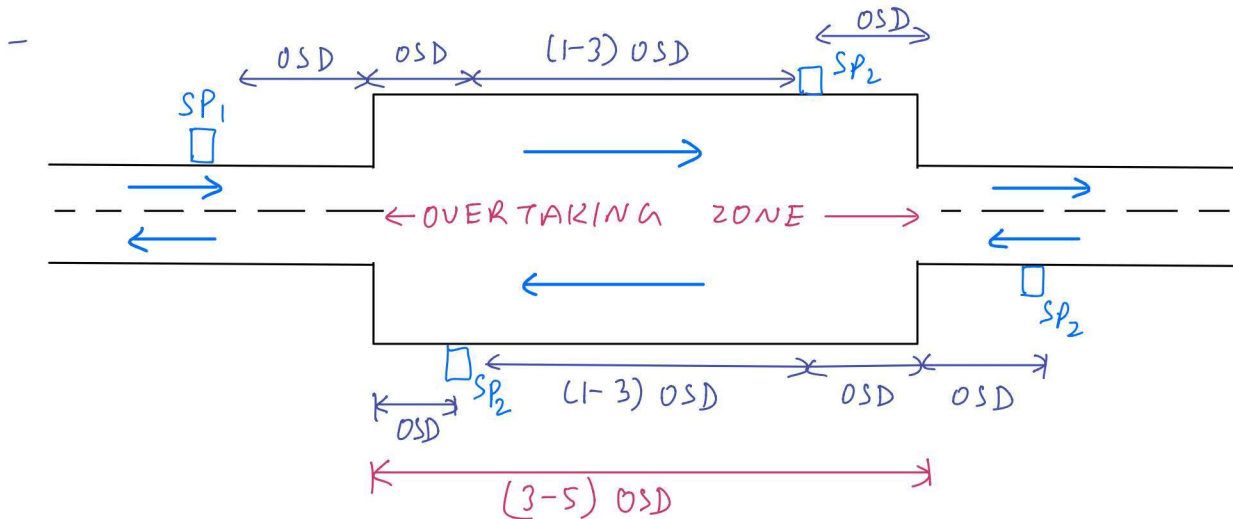
- On road stretches with one way traffic.

$$OSD = d_1 + d_2$$

- On divided highway with four or more lanes. $(d_1 + d_2)$
it is not essential to provide the usual OSD. However the SD on any highway should be more than SSD.

OVERTAKING ZONE :-

- If OSD cannot be provided over a particular stretch of road section in such cases, for same overtaking of the vehicle at design speed opportunity must be given to the vehicle, such zones are termed as "OVERTAKING ZONES".



SP_1 = Sign post "OVERTAKING ZONE AHEAD".

SP_2 = Sign post "END OF OVERTAKING ZONE".

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- For two way traffic, $OSD = d_1 + d_2 + d_3$

- For one way traffic, $OSD = d_1 + d_2$.

NOTE:- If at stretches of road OSD cannot be provided as per suggested by IRC in such case sight distance is desirable to be kept as $2 \times SSD$ and it is termed as "Intermediate SD".

$$ISD = 2 \times SSD \rightarrow \text{Use object height} = 1.2 \text{ m.}$$

2) The measurement of ISD can be made assuming both the height of eye of driver and that of object to be 1.2 m

DISTANCE (m)	Reaction Time (Sec)	HEIGHT (m)	
		Driver eye	object
SSD	2.5	1.2	0.15
OSD	2	1.2	1.2
ISD	2.5	1.2	1.2
Space head way	0.7	-	-

Magnitude :-

$$OSD > ISD > SSD$$

- 8) The speeds of overtaking & overtaken vehicles are 70 and 40 kmph respectively on a two way traffic road. The avg accel during overtaking may be assumed may be 0.99 m/sec^2 .
calculate

(a) Safe OSD

(b) what is min length of overtaking zone.

(c) what should be the value of headlight SD.

Sol (a) $V_A = 70 \text{ kmph}$ $a = 0.99 \text{ m/s}$
 $V_B = 40 \text{ kmph}$

$$\begin{aligned} OSD &= 0.278 V_B t_n + 2s + b + d_3 \quad S = 0.7 V_B + b \\ &= 0.278 \times 40 \times 2 + 82.9 + 2 \times 13.784 = 0.7 \times 0.278 \times 40 + b \\ &= 132.708 \text{ m} + 145.71 \quad b = V_B T \quad T = \sqrt{\frac{4S}{a}} \\ &= 278 \text{ m.} \quad b = 82.9 \text{ m} \quad T = \sqrt{\frac{4 \times 13.784}{0.99}} \end{aligned}$$

(b) min length = $3 \times 278 = 834 \text{ m.}$

(c) head light SD = SSD at night

$$SSD = 0.278 \times V_d t_n + \frac{V_d^2}{254 f}$$

Assume, $f = 0.36$ ($V = 70 \text{ kmph}$)

$$SSD = 0.278 \times 70 \times 2.5 + \frac{70^2}{254 \times 0.36} =$$

$$SSD = 102.23 \text{ m}$$

Jan 2

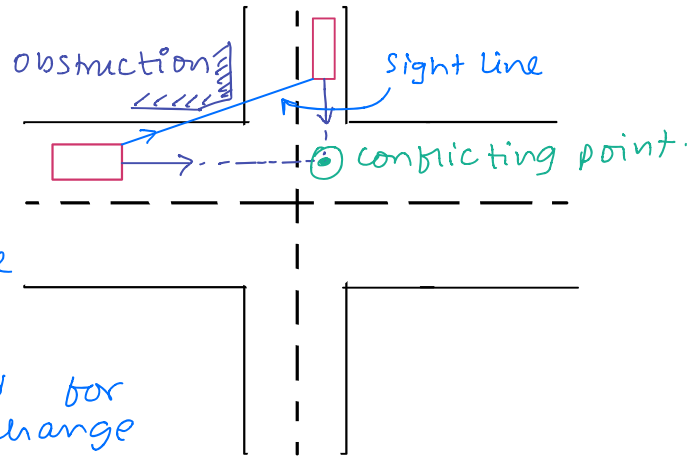
(IV) SIGHT DISTANCE AT INTERSECTION :-

(a) UNCONTROLLED INTERSECTION :-

- On all approaches of intersecting roads there should be clear view across the corner from the sufficient distance so as to avoid the collision
- The design sight distance at intersection may be based on three possible conditions.

- (a) enabling the approaching vehicle to change the speed (slow down).
- (b) Enabling the approaching vehicle to stop.
- (c) Enabling stopped vehicle to cross the road.

(i) The SD should be sufficient to enable either one or both approaching vehicle to change the speed to avoid collision.



- The vehicle approaching from the minor must slow down
- Total reaction time required for the driver to decide to change speed is 2sec. & atleast 1 more sec is required for making the change in speed.

$$\therefore \text{Total time} = 2 + 1 = 3 \text{ sec}$$

$$\text{Hence } SD = 0.278 V_d \times t_r = (0.278 \times V_d \times 3) \text{ m.}$$

↳ (kmph)

(ii) The distance for the approaching vehicle should be sufficient to bring either both or one of the vehicle before reaching the conflict.

- hence it will be equal to SSD.

(iii) In this case vehicle entering the intersection from the minor roads are controlled by stop or give away sign.

so the vehicles have to stop and then proceed to cross the main road.

- In this case the total time required by the stopped vehicle would include to start accelerate and cross the main road before another vehicle travelling at design speed approaches it from main road.

For safety consideration, the sight distance at uncontrolled intersection is maximum of above three cases.

(b) CONTROLLED INTERSECTION :-

- At signalised intersections, the above three requirements are not applicable.
- At rotary intersection the sight distance should be atleast equal to SSD for design speed of rotary.

Jan 4 (III) HORIZONTAL ALIGNMENT OF HIGHWAY

- change in direction are required in highway alignment due to the various reasons.

- (a) Topographic condition
- (b) obligatory points

- various design element to be considered in the horizontal elements are

- (i) DESIGN SPEED.
- (ii) RADIUS OF CURVE.
- (iii) TYPE & LENGTH OF TRANSITION CURVE.
- (iv) WIDTH OF PAVEMENT.
- (v) SUPER ELEVATION.
- (vi) SET BACK DISTANCE.

DESIGN SPEED :- The design speed is the main factor on which most of the geometric design elements depends including horizontal elements.

- The design speed of road depends upon class of road and terrain
- classification of terrain & design speed for different class of road is as follows.

TERRAIN CLASSIFICATION	CROSS SLOPE (%)
PLAIN	0-10
ROLLING	10-25
MOUNTAINOUS	25-60
STEEP	above 60.

Type of Road	DESIGN SPEED							
	PLAIN		ROLLING		MOUNTAIN		STEEP	
	Rolling	Min.	Rolling	Min.	Rolling	Min.	Rolling	Min.
Express way	120	100	100	80	80	60	80	60
NH, SH	100	80	80	65	50	40	40	30
MDR	80	65	65	50	40	30	30	20
ODR	65	50	50	40	30	25	35	20
VR	50	40	40	35	25	20	25	20

- Design speeds are also being defined for urban roads.
- Two values of the speed i.e. rolling and min. are being specified for each type of road.
- As far as possible an attempt should be made to design all the geometric elements for rolling speed however if site conditions and economic consideration doesnot permit so, designing must be done for min design speed.

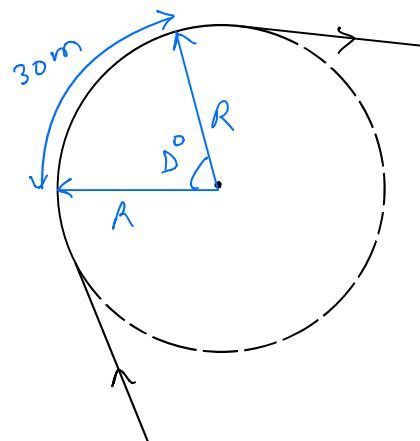
HORIZONTAL CURVE :-

- It is a curve in plan to provide change in direction to centre line of the road.
- It may be defined in terms of
 - (a) Radius of the curve. (m)
 - (b) Degree of the curve :- Degree of the curve is the central angle subtended by an arc of length 30m.

Angle	length
360°	$2\pi r$
1°	$\frac{2\pi r}{360}$

$$1^\circ \rightarrow \frac{\pi r}{180}$$

$$D^\circ - \frac{\pi r D}{180} = 30m$$



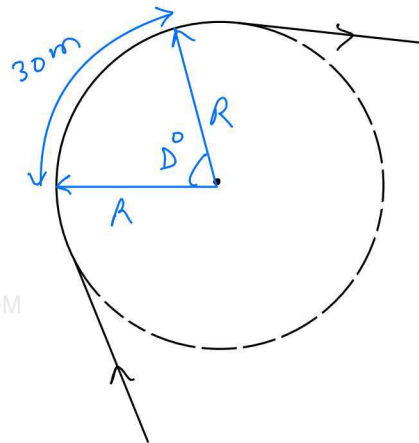
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Angle	length
360°	$2\pi r$
1°	$\frac{2\pi r}{360}$
$1^\circ \rightarrow$	$\frac{\pi r}{180}$
$D^\circ -$	$\frac{\pi r}{180} D = 30m$

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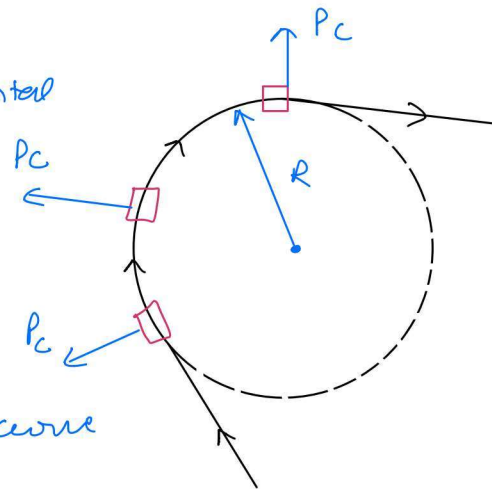


$$\therefore R = \frac{1718.8}{D} \approx \frac{1720}{D}$$

$$R = \frac{1720}{D^\circ}$$

- When a vehicle travels on horizontal curve, the centrifugal force acts horizontally outwards through the CG of the vehicle.

- This centrifugal force depends upon speed of vehicle on a radius of horizontal curve

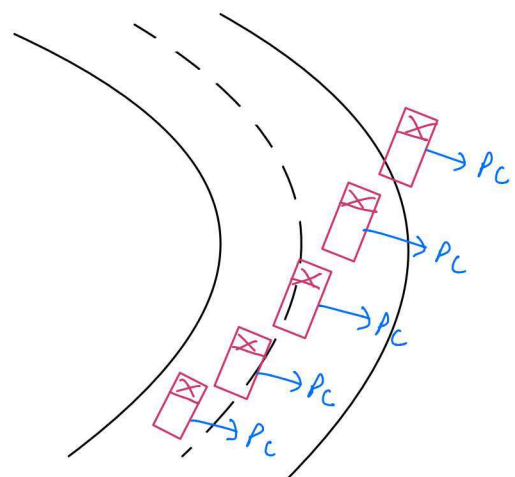
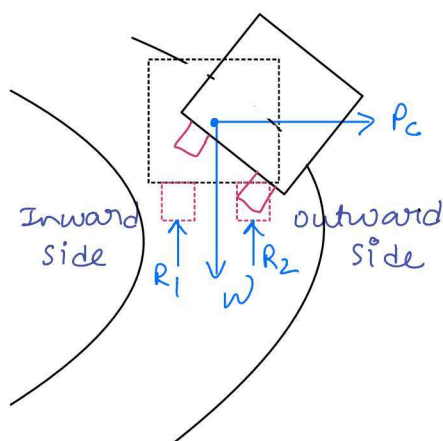


- It will be counteracted by lateral/transverse friction developed b/w the tyre and pavement which helps the vehicle to change direction along the curve.

$$P_c = \frac{mv^2}{R} = \frac{wv^2}{gR}$$

here, $\frac{P_c}{w} = \frac{v^2}{gR}$ is termed as **IMPACT FACTOR OR CENTRIFUGAL RATIO**

- The force acting on vehicle has the tendency to either overturn the vehicle outwards about wheels & to skid the vehicle laterally outwards.



- For stability of vehicle both the conditions are to be checked as follows.

(i) OVERTURNING EFFECT

For no overturning

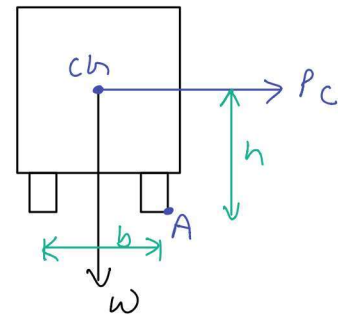
$$M_R \geq M_O$$

$b =$ 4c dist. of tyre

$$M_R = W \cdot \frac{b}{2} \geq P_C \cdot h$$

$$\frac{P_C}{W} \leq \frac{b}{2h} \Rightarrow \text{condition to prevent overturning}$$

$$\boxed{I.F \leq \frac{b}{2h}}$$



$\Rightarrow$ If $I.F = b/2h$, vehicle is on verge of overturning.

$\Rightarrow$ If $I.F > b/2h$, vehicle will overturn.

(ii) TRANSVERSE SKIDDING EFFECT :-

For no-skidding

$$F_A + F_B \geq P_C$$

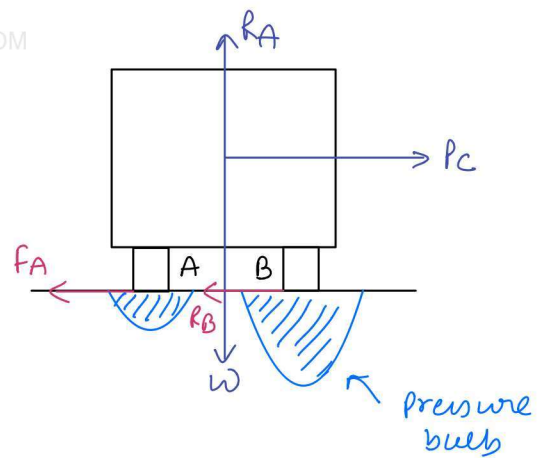
$$\mu R_A + \mu R_B \geq P_C$$

$$\mu (R_A + R_B) \geq P_C$$

$$\mu W \geq P_C$$

$$\frac{P_C}{W} \leq \mu \approx f \rightarrow \text{transverse direction}$$

$$\boxed{I.F \leq \mu \approx f}$$

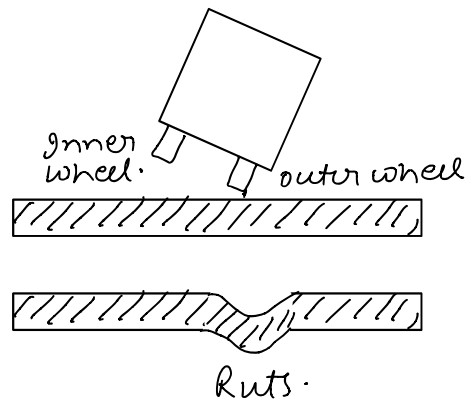


NOTE:- If pavement is kept horizontal across the alignment the pressure on the outer wheel will be higher due to P_C acting outward.

1) when the limiting condition for overturning occurs the pressure on the inner wheels becomes 0.

2) Since pressure on outer wheel is comparatively more it damages the pavement by developed ruts over it.

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- hence to avoid both overturning & sliding

$$\frac{P_c}{w} \leq \min \left\{ \begin{array}{l} \frac{b}{2h} \\ f \end{array} \right.$$

- The relative danger of lateral skidding & overturning depends upon whether "f" is lower or higher than " $\frac{b}{2h}$ ".

If $f < \frac{b}{2h} \Rightarrow$ Transverse skidding before overturning.

$f > \frac{b}{2h} \Rightarrow$ overturning before transverse skidding.

Jan 5 # SUPER ELEVATION

- In order to counteract the effect of centrifugal force and to reduce the tendency of the vehicle to overturn and skid, outer edge of pavement is raised wrt inner edges, thus providing a transverse slope throughout the length of the horizontal curve.
- This transverse inclination to the pavement surface is known as super elevation (CANT) BANKING

$\Sigma F_x = 0 :-$

$$w \sin \theta + F_1 + F_2 = P_c \cos \theta$$

$\Sigma F_y = 0 :-$

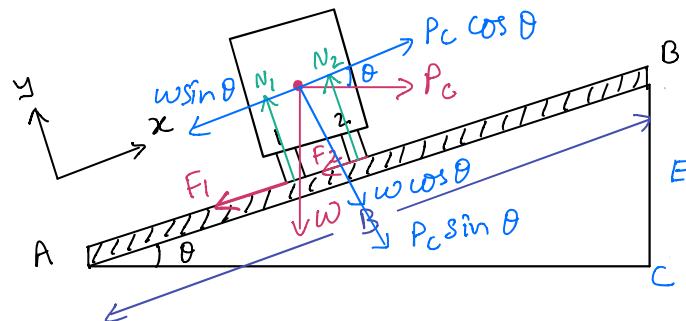
$$R_1 + R_2 = w \cos \theta + P_c \sin \theta$$

$e/\theta/se/\tan \theta/\sin \theta \rightarrow$ super elevation.

$E =$ Total super elevation height of outer edge,

The total rise of outer edge, $E = eB$

Height (m/cm/mm) $\leftarrow$ super elevation (degree, rad, 1:n)



$$\tan \theta = \frac{BC}{AC}$$

if θ is small, $\theta \approx \sin \theta \approx \tan \theta$

$$\sin \theta = \frac{BC}{B} \quad e = \frac{BC}{B}$$

Analysis of Superelevation :-

Forces acting on vehicles are

- (i) Self weight (w) $\rightarrow$ vertically downward.
- (ii) centrifugal force (P_c) $\rightarrow$ Acting horizontally in outward direction.
- (iii) frictional force (F) $\rightarrow$ laterally inward along the pavement surface

For equilibrium condition :-
 $\sum F_x = 0 :-$ $\sum F_y = 0 :-$

$$w \sin \theta + F_1 + F_2 = P_c \cos \theta \quad \text{--- (i)}$$

$$R_1 + R_2 = w \cos \theta + P_c \sin \theta \quad \text{--- (ii)}$$

From (i) & (ii)

$$P_c \cos \theta = w \sin \theta + f (w \cos \theta + P_c \sin \theta)$$

$$P_c (\cos \theta - f \sin \theta) = w (\sin \theta + f \cos \theta)$$

Impact factor / centrifugal ratio $\leftarrow \left(\frac{P_c}{w} \right) = \frac{\sin \theta + f \cos \theta}{\cos \theta - f \sin \theta}$

when, $\theta(e)$ is small, $\sin \theta \approx \tan \theta$, $\cos \theta \approx 1$.

$$\frac{P_c}{w} = \frac{\tan \theta + f}{1 - f \tan \theta}$$

$$\frac{P_c}{w} = \frac{e + f}{1 - ef}$$

here, $e_{\max} = 10\% = 0.1$ (extreme case) (normally $e = 7\%$)

and $f_{\min} = 0.10$, $f_{\text{normal}} = 0.15$

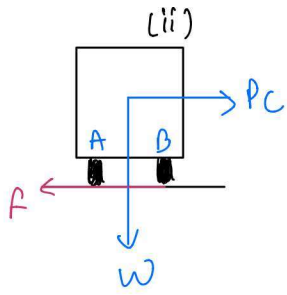
$$1 - ef = 0.9998 \approx 1 \rightarrow \text{As per IRC you can take } \underline{\underline{1 - ef = 1}}$$

$$\frac{P_c}{w} = e + f \quad \text{and} \quad \frac{P_c}{w} = \frac{v^2}{gR}$$

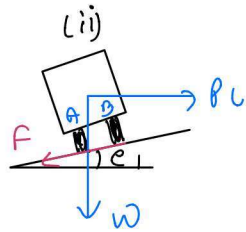
$$e + f = \frac{v^2}{gR} \quad (m/sec)$$

$$e + f = \frac{v^2}{127R} \quad \rightarrow \text{Kmph}$$

R = Radius of curve (m)
 v = velocity (Kmph).



$$R_B > R_A$$

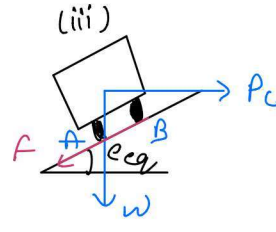


$$e_1 < e_{eq} < e_2$$

$$R_{B(ii)} < R_{B(i)}$$

$$R_{A(ii)} > R_{A(i)}$$

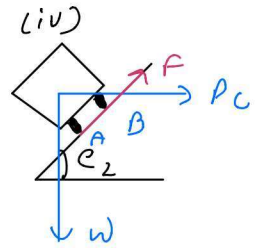
$$R_B > R_A$$



$$R_{B(iii)} < R_{B(ii)}$$

$$R_{A(iii)} > R_{A(ii)}$$

$$R_A = R_B$$



$$R_{B(iv)} < R_{B(iii)}$$

$$R_{A(iv)} > R_{A(iii)}$$

$$R_B < R_A$$

NOTE:- It may be seen that the contribution of super elevation "e" (0.07) is much less than that of lateral friction "f" (0.15) in counteracting centrifugal force.

2) In some situations eg at intersection or some obstruction it is not possible to provide super elevation, in such cases friction counteracts the centrifugal force & if friction is also not sufficient then radius of curve is increased, which if not possible, speed of vehicle negotiating a turn must be reduced to following.

$$e + f = \frac{v^2}{127R}$$

$$e = 0$$

$$v_a = \sqrt{127fR}$$

v_a = allowable speed.

3) In some case, super elevation may be (-)ve, in such case

$$f - e = \frac{v^2}{127R}$$

∴ hence, $e = f(v, R, f)$

8) The radius of a horizontal curve is 110m, the design speed is 60 kmph & $f=0.15$

(a) calculate the super elevation req if full lateral friction is assumed to develop.

(b) calculate the value of f_L needed if no e is provided.

Sol
 $R = 110\text{m}$ $f = 0.15$
 $V = 60\text{ kmph}$

(a) $e + f = \frac{V^2}{127R}$

$e + 0.15 = \frac{3609}{127 \times 110} = 0.257 \rightarrow \text{required}$

$e = 0.107$

(b) $e = 0, \quad f = \frac{60^2}{127 \times 110} = 0.257 \rightarrow \text{required}$

- From previous analysis the value of e needed increases with increase in speed & with decrease in radius of curve.

- Hence it(e) cannot be incr. beyond a particular limit, as it would lead to the toppling / instability of slow moving vehicle with high COG.
 eg $\rightarrow$ Bullock carts, Truck, lorry.

- value of e_{max} for different terrain as follows.

TERRAIN TYPE	$e_{\text{max}}(\%)$
1) plain & rolling	7%
2) Mountainous & steep not bounded by snow.	10%
3) Urban / Builtup area	4%

- From drainage consideration it is necessary to have minimum cross-slope to drain out surface water.

if $e_{\text{calculated}} < \text{cross-slope of road}$

- Then cross-slope may be provided as super-elevation only. by eliminating the crown and negative super-elevation on outer edge.
- In very flat curve with large value of P_c & negative super-elevation may be permitted on outer edge.
- As per IRC, radius of the curve beyond which "e" is not required is as follows:-

Design Speed (km/h)	R (m) of horizontal curve				
	4%	3%	2.5%	2%	1.7% → camber
20	50	60	70	90	100
25	70	90	110	140	150
30	100	130	160	200	240
35	140	180	220	270	320
40	180	240	280	350	420
50	280	370	450	550	650
60	470	620	750	950	1100

Jan 6

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DESIGN OF SUPER ELEVATION :-

- Since mixed traffic operates on our road, the super-elevation design is very complex.
- Providing a lower value of super elevation and relying on lateral friction would be unsafe for fast moving vehicle and providing higher value of super elevation creates problem for slow moving vehicle.
- Hence a compromised value for "e" is designed as follows.

Step 1:- The super-elevation is calculated for 75% of design speed, neglecting the lateral friction.

$$e + f = \frac{v^2}{127R}$$

For design, $f = 0$, $v_{\text{des}} = 0.75v$

$$e = \frac{(0.75v)^2}{127R}$$

$$e = \frac{v^2}{225R}$$

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Step 2:- If the calculated value of e is less than 7%, so obtained " e " is provided.

- If value of " e " obtained is more than 7%, then provide " e " = 7% and check for lateral friction only. (plain & rolling)

Step 3 :- check for friction developed for maximum value of $e = 7\%$ at full value of design speed.

$$e + f = \frac{v^2}{127R}$$

$$f = \frac{v^2}{127R} - e$$

$$f = \frac{v^2}{127R} - 0.07$$

If the value of " f " calculated is less than 0.15, the super elevation of 7% is safe for design but if comes out to be more than 0.15, then the speed of vehicle has to be restricted to safe value which is less than design speed.

Step 4 :- The allowable speed at the corner is computed as follows.

$$e + f = \frac{v^2}{127R}$$

$$0.07 + 0.15 = \frac{v_a^2}{127R}$$

$$v_a = 5.28 \sqrt{R}$$

Kmph m.

- If the allowable speed is higher than the design speed, then the design is adequate, but if it is less than design speed, the speed is limited to the allowable speed.
- Speed can be reduced by warning sign or by speed limit regulation signs.
- on Imp highway speed is not reduced, hence the curve should be realigned with larger radius radius of curvature.

Q) The design speed of highway is 80 kmph. There is a horizontal curve of radius 200m. On a certain locality safe limit of $f = 0.15$

(a) calculate the e required to maintain this speed.

(b) If max e of 7% is not to be exceeded, compute the max allowable speed on this horizontal curve as it is not possible to redesign.

Sol
 $v_d = 80 \text{ kmph}$
 $R = 200 \text{ m}$
 $f = 0.15$

(a) $e = \frac{v^2}{225R}$
 $e = \frac{80^2}{225 \times 200} = 0.142 \text{ (7.7\%)}$

(b) $e = \frac{v^2}{225R}$
 $e = \frac{80^2}{225 \times 200}$
 $e = 0.142 = 14.2\% \text{ (7.7\%)}$

$0.07 + f = \frac{80^2}{127 \times 200}$
 $f = 0.1819 \text{ (>0.15)}$

$\therefore v_a = 5.28 \sqrt{100}$
 $= 74.67 \text{ km/hr.} = 70 \text{ km/h}$

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Q) A MDR with thin bituminous pavement surface in low rainfall area has horizontal curve of radius 1400 m. If the design speed is 65 kmph, what should be the super elevation.

Sol
 $e = \frac{v^2}{225R} = \frac{65^2}{225 \times 1400} = 1.34\% \text{ (0.0134)}$

But the value of camber required for drainage is 2%.

$e < 2\%$ $\therefore$ provide 2% of super elevation.

We need to check for negative "e" on outer edge of curve

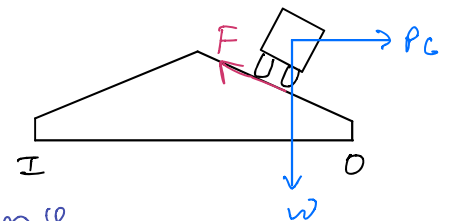
$f - e = \frac{v^2}{127R}$

$f - e = 0.15 - 0.02 = 0.13 \Rightarrow \text{Resistance}$

and $\frac{v^2}{127R} = \frac{65^2}{127 \times 1400} = 0.023 \Rightarrow \text{CF ratio.}$

Here, $-e + f > \frac{v^2}{127R}$

Hence vehicle is safe in negative super elevation also

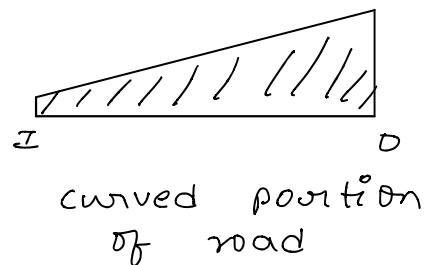
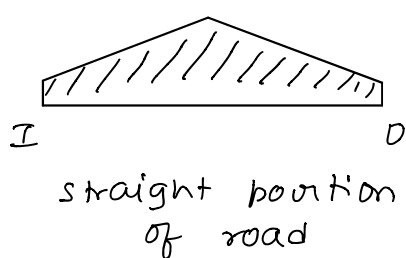


Thus, we can safely provide 2%.

NOTE:- on outer edge friction mobilised would be less than f .

Attainment of super elevation in the field :-

- Introduction of super elevation on horizontal curve in the field is very typical as on straight portion of road cross-section normal camber is provided with crown at the centre, sloping downward on both the sides.
- But the portion of q_s on circular curve is super elevated uniformly sloping down from outer edge upto the inner edge of the pavement.



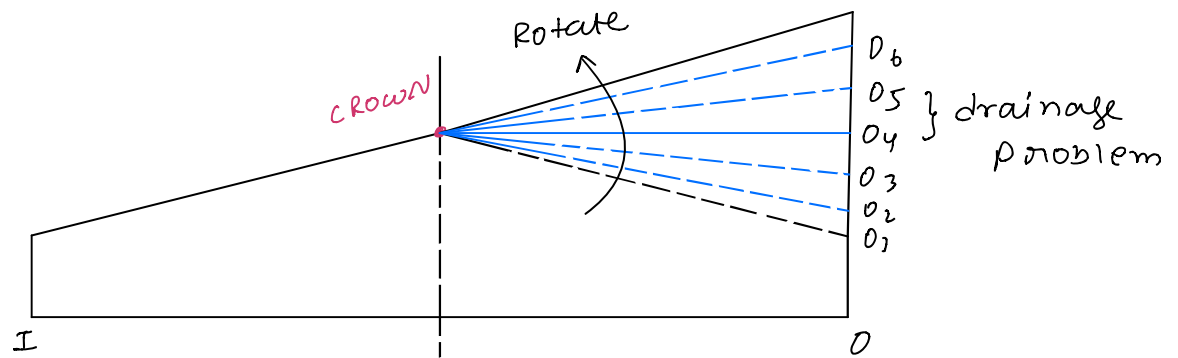
- The attainment of super elevation is done in two stages.

- (i) Elimination of the crown of camber section
- (ii) Rotation of pavement to attain full super elevation.

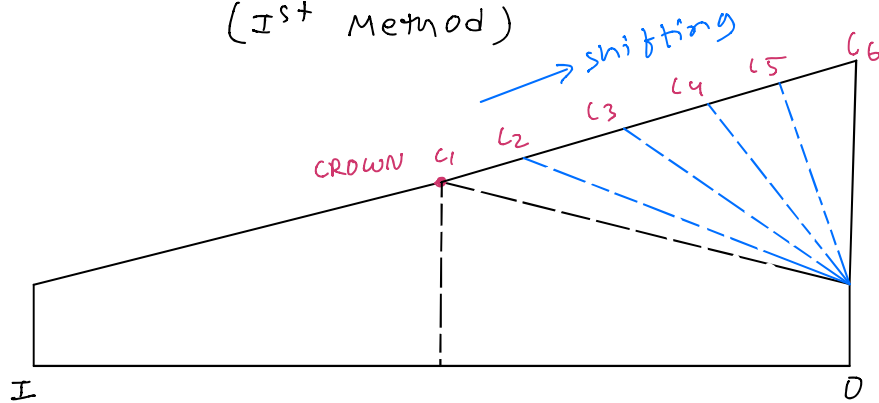
Stage (i) : Elimination of crown of cambered section.

- It can also be achieved by two methods.

- (a) In first method outer half of the cross-slope is rotated about the crown at desired rate, to attain the super elevation of inner half and c -line is not altered.
- (b) In 2nd mtd (DIAGONAL CROWN METHOD) crown is shifted progressively outward, thus increasing the width of the inner half of c/s .
→ negative super elevation is increased which is not safe for slow moving vehicle.



(1st Method)

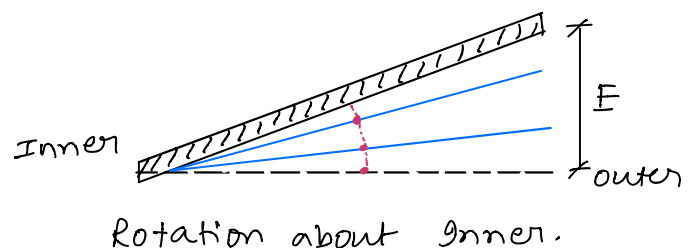
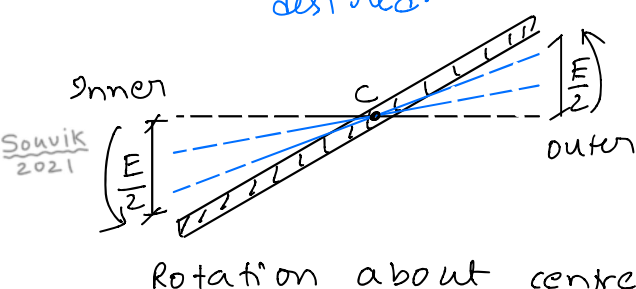


Qam 7 Stage (ii) :- Rotation of pavement to attain full super-elevation.

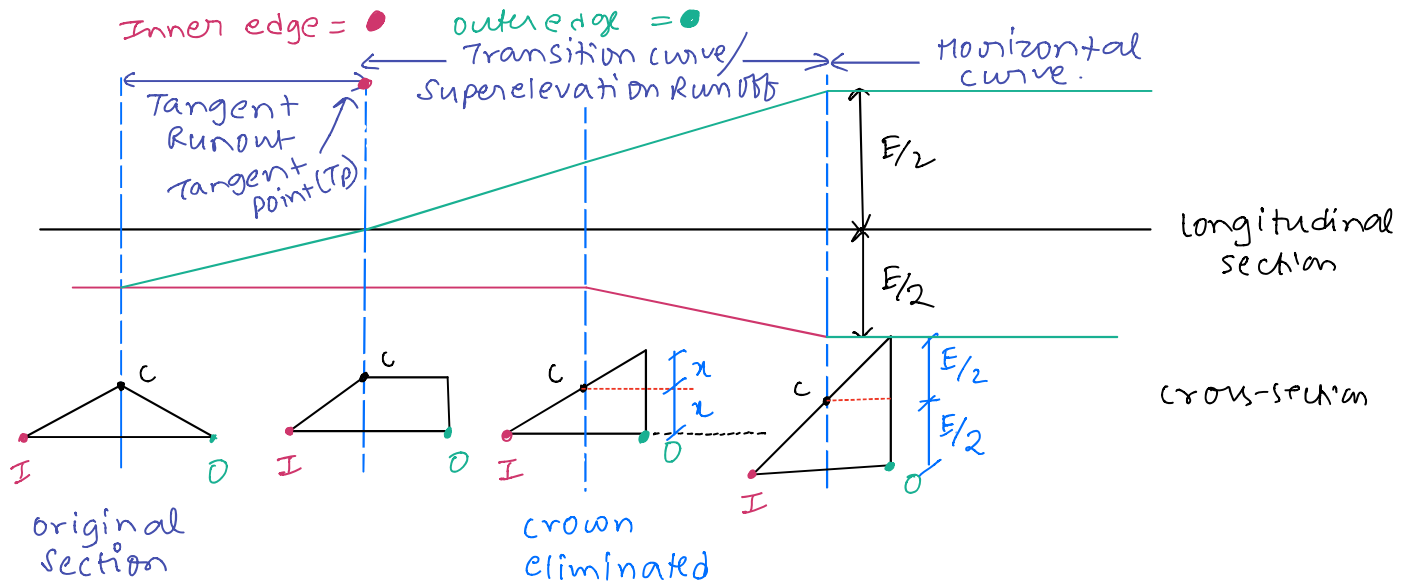
- When the crown of the camber is eliminated, it does not signify that super-elevation desired at that section is attained. ESENOTES.COM
- If desired super-elevation is more than camber / cross-slope, the pavement section is rotated further till the desired elevation is attained.
- Desired Super-elevation can be attained by two mtd.

(a) By rotating the pavement about the centre line depressing the inner edge and raising the outer edge each by half the total amount of super-elevation required.

(b) By rotating the pavement about inner edge of the pavement raising both centre as well as the outer edge of pavement such that the outer edge is raised by full amount of super-elevation desired.



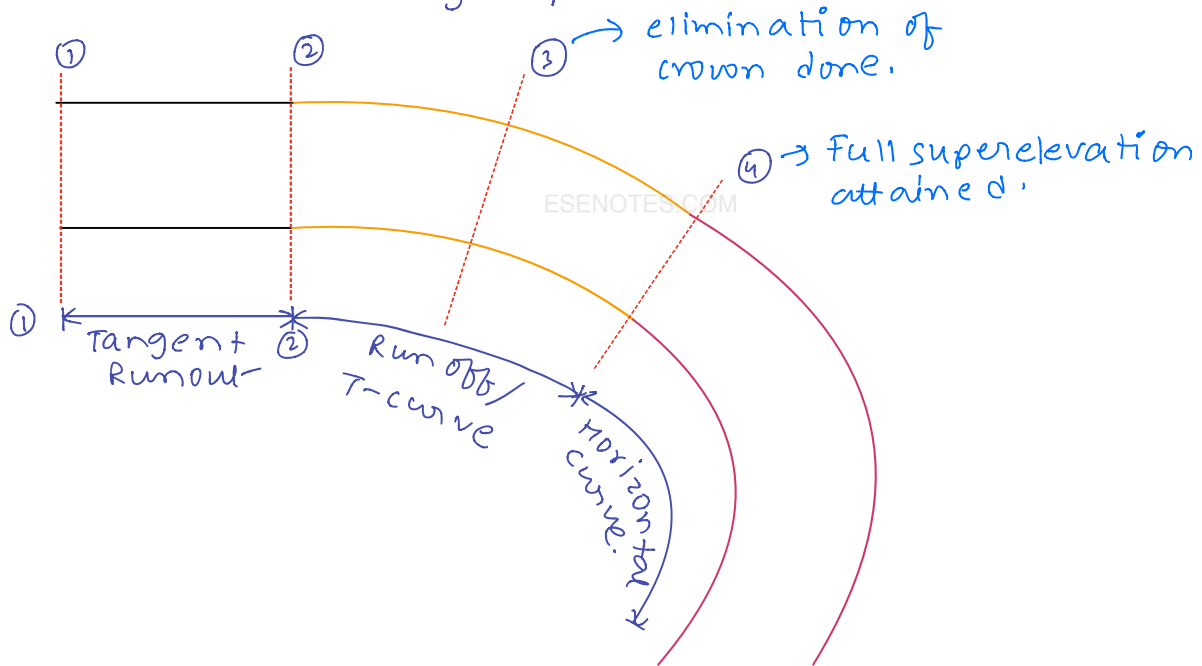
(a)



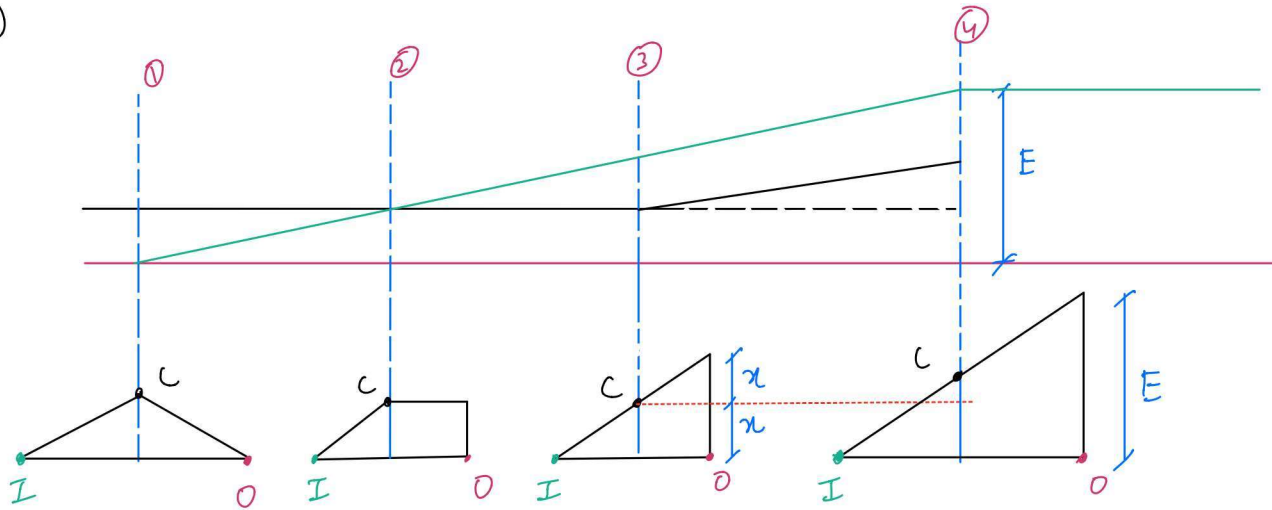
Stage (I): Elimination of crown

Stage (II): Attainment of super elevation.

→ From 1-3 → drainage problem will occur.



(b)



Rotation about centre line

- (i) It may interfere with drainage system
- (ii) Centre line remains unchanged
- (iii) Earthwork is balanced in filling & cutting

Rotation about inner edge.

- (i) No interference with drainage system.
- (ii) C-line is elevated
- (iii) Filling is required.

- In cases where T-curve cannot be provided for some reason, $\frac{2}{3}$ rd of super elevation may be attained at straight portion before the start of the circular curve & the balance $\frac{1}{3}$ is provided at the beginning of curve.

- The super elevation is introduced by the raising the outer edge of pavement - at a rate not exceeding (1 in 150) in plain & rolling and (1 in 60) in mountainous & steep terrain.

RADIUS OF HORIZONTAL CURVE :-

- horizontal curve of highways are designed for specified design speed (Ruling & minimum)
- however if this is not possible due to site condition then horizontal curve may be redesigned.

Souvik 2021 - On the basis of speed it is classified into following.

(a) Rolling minimum Radius :- (RMR).

$$e + f = \frac{v^2}{127(RMR)}$$

$$(RMR) = \frac{v^2}{(e_{max} + f) 127}$$

v = rolling design speed, (kmph)

(b) Absolute minimum Radius :- (AMR)

$$e + f = \frac{v^2}{127(AMR)}$$

$$(AMR) = \frac{v^2}{127(e_{max} + f)}$$

v = Min. design speed (kmph)

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(c) Radius beyond which no SE is required (RBSN).

- It the radius corresponding to which camber serves the purpose of super elevation.

$$e + f = \frac{v^2}{127R}$$

e = camber, $f = 0$. $v = 75\% v_d$

$$\text{camber} = \frac{(0.75v)^2}{127R}$$

$$R_{RBSN} = \frac{v^2}{225 \cdot \text{camber}}$$

Q) calculate the values of rolling min radius and absolute min radius of horizontal curve of a National highway in plain terrain.

Sol

(i) $RMR = \frac{v^2}{127(e_{max} + f)}$ $v \rightarrow$ rolling design speed

For, NH $v = 100$ kmph

$$RMR = \frac{100^2}{127(0.07 + 0.15)} = 358 \text{ m}$$

(ii) $AMR = \frac{v^2}{127(e_{max} + f)} = \frac{80^2}{127(0.07 + 0.15)} = 229 \text{ m}$

NH, $v = 80$ kmph

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2021

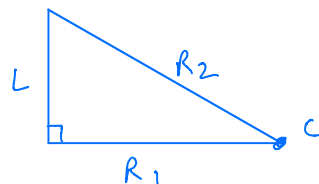
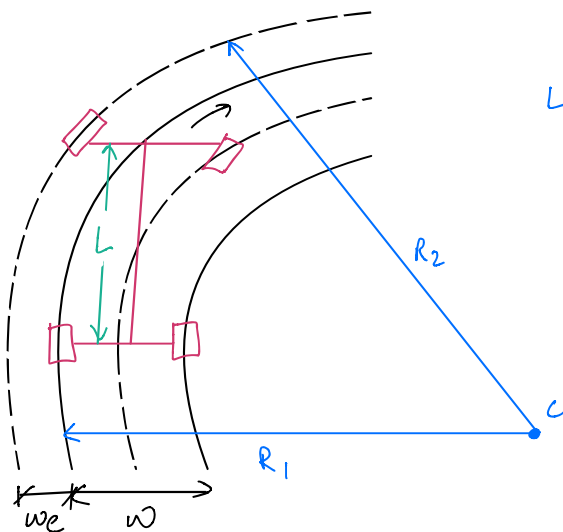
Minimum Radius of Horizontal curve for different terrain are as follows.

Type of Road	Plain terrain		Rolling terrain		Mount. Terrain		steep terrain	
	RMR	AMR	RMR	AMR	not affected by snow.			
					RMR	AMR	RMR	AMR.
NH or SH	360	230	230	155	80	30	50	30
MDR	230	155	155	90	50	30	30	14
ODR	155	90	90	60	30	60	20	14
UR	90	60	60	45	20	14	20	14

WIDENING OF PAVEMENT ON H-CURVE :-

- On horizontal curve, which are not of very large radius, common practice is to widen the pavement slightly more, than the normal width.
- The object of providing this extra widening on horizontal curve is as follows :-

(a) To avoid the offtracking of vehicle.



- An automobile has a rigid and only the front wheels can be turned, hence when the vehicle takes a turn to negotiate horizontal curve, the rear wheel do not follow the same path as that of front wheel.

- If no lateral skidding of rear wheels takes place, the rear wheels follows the inner path on the curve as compared to front wheels.

- This implies if inner front wheel takes a path on the inner edge of pavement on horizontal curve, the inner rear wheel will be off the pavement of inner shoulder and viceversa.

(b) To encounter psychology of the driver.

- To maintain clearance b/w the opposite moving vehicle or overtaking vehicle

(c) To avoid lateral skidding.

(d) To increase the visibility at the curve.

note:- Extra widening is a function of [length of wheel base (L), Radius of curve (R), Psychological factor]

The extra widening of the pavement on horizontal curve is divided into two parts.

(a) Mechanical widening

(b) psychological widening

→ To prevent the off tracking of the vehicle as follows.

$$R_2^2 = R_1^2 + L^2$$

also, $R_2 - R_1 = w_m$

$$R_2^2 = (R_2 - w_m)^2 + L^2$$

$$R_2^2 = R_2^2 - 2R_2w_m + w_m^2 + L^2$$

$$w_m (2R_2 - w_m) = L^2$$

$$w_m = \frac{L^2}{2R_2 - w_m}$$

$$2R_2 \gg w_m \text{ thus, } 2R_2 - w_m \approx 2R \quad R = \text{Mean Radius of curve.}$$

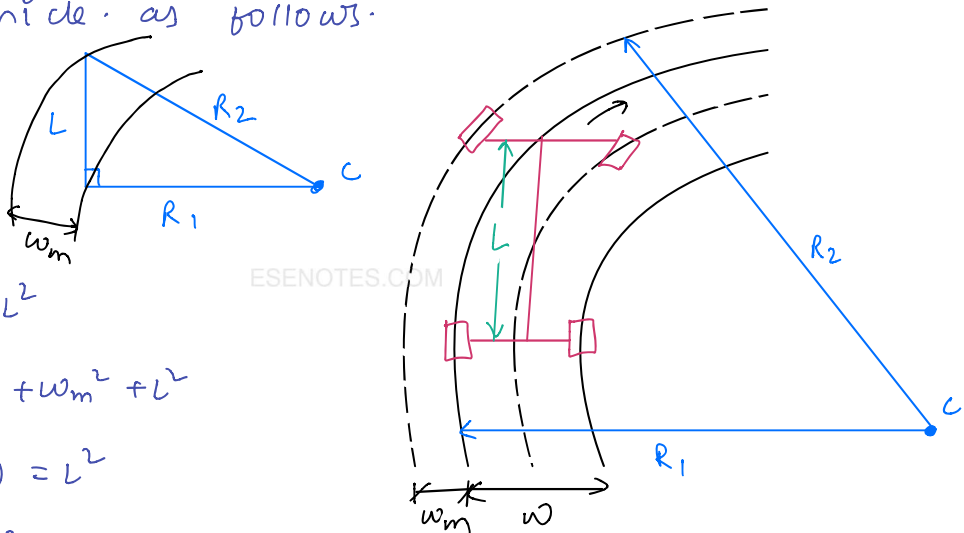
$$w_m = \frac{L^2}{2R}$$

For "n" lane roads,

$$w_m = \frac{nL^2}{2R}$$

NOTE:- Hence in a road having "n" traffic lanes as "n" vehicles can travel simultaneously, w_m required would be.

length of wheel base, $L = 6.1 \text{ m}$



- (b) At horizontal curve drivers have the tendency to maintain a greater clearance b/w the vehicles than a straight stretch of road. Therefore an extra width is req for this, which is given empirically by following relation

$$w_p = \frac{V}{9.5\sqrt{R}}$$

$V \rightarrow$ design speed (kmph)
 $R \rightarrow$ Radius (m)

need not to convert.

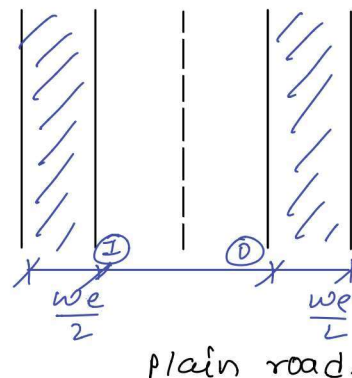
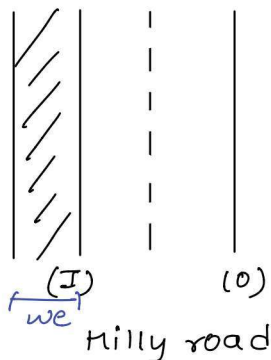
Hence total widening, $w_e = w_n + w_p$

$$w_e = \frac{n d^2}{2R} + \frac{V}{9.5\sqrt{R}}$$

NOTE:-

- For single lane road and two lane road if radius $> 300\text{m}$ then extra-widening is not required
- For single lane road, psychological widening is not considered. ($w_p = 0$).
- Extra widening is provided gradually through transition curve.
- Generally extra widening is provided equally on both the sides of curve.

But on hilly roads w_e is provided only on inner side.

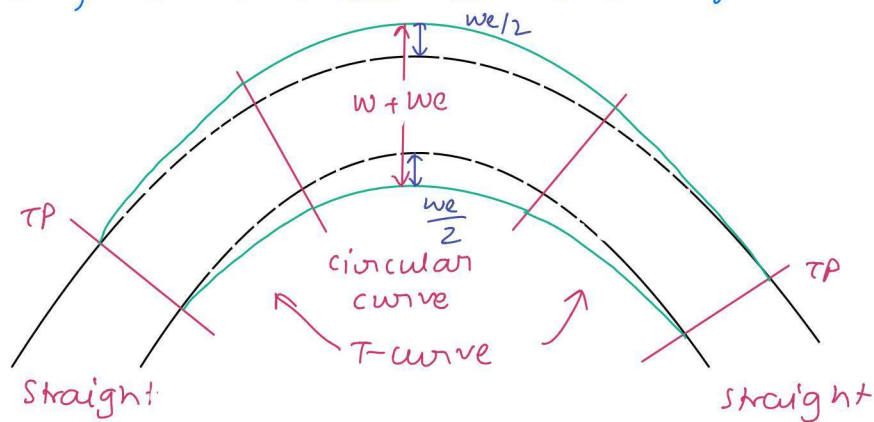


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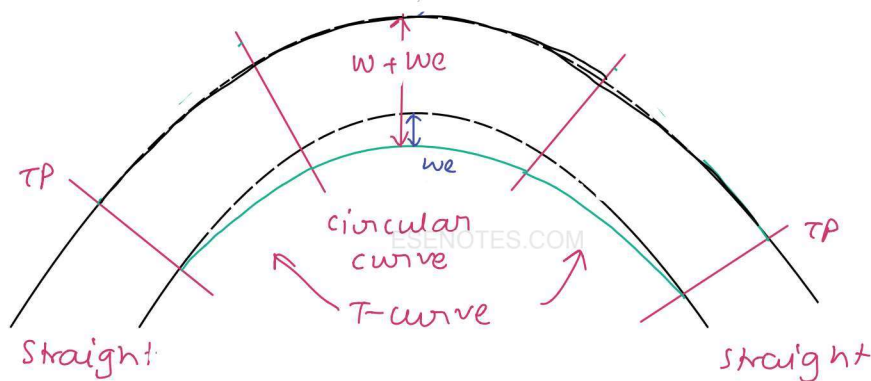
- Extra width of the pavement at horizontal curve are as follows.

Radius of curve (m)	≤ 20	26-40	41-60	61-100	101-300	> 300
Extra width on two lane road	1.5	1.5	1.2	0.9	0.6	—
Extra width on single lane road	0.9	0.6	0.6	—	—	—

- The widening is introduced gradually starting from the tangent point i.e. beginning of T-curve at uniform rate equally on both sides in plain terrain and on inner side on hilly terrain such that designed widening is attained at end of the transition curve.



plain terrain Highway.



Hilly terrain highway.

- On Horizontal circular curve without T-curves, $\frac{2}{3}$ rd of the widening is provided at the end of the straight portion and remaining $\frac{1}{3}$ rd widening is provided on circular curve.

Q) Find the total width of pavement on a H-curve for a new N.H. to be aligned along a rolling terrain with rolling min. radius.

Sol For N.H., Rolling min. radius on a rolling terrain is

$R_{MR} = 230 \text{ m} \rightarrow$ can be directly assumed.

For N.H., Rolling terrain, $RMS = 80 \text{ kmph}$.

Normal width of pavement = 7m (2 lanes)

$L = 6 \text{ m}$, $e = 7\%$, $f = 0.15\%$

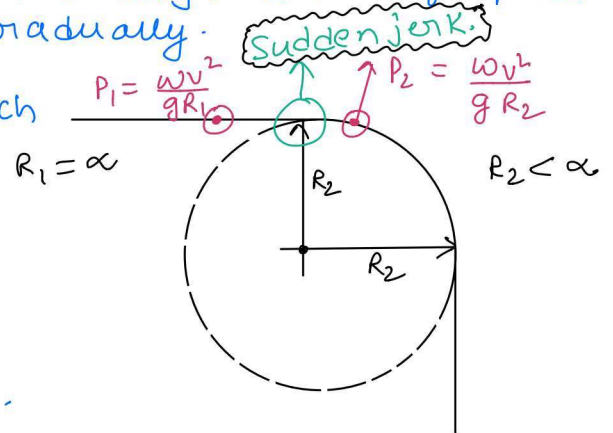
$$R_{\text{RMR}} = \frac{v^2}{127(e_{\text{max}} + f)} = \frac{80^2}{127(0.07 + 0.15)} = 230 \text{ m}$$

$$w_e = w_m + w_p = \frac{nl^2}{2R} + \frac{v}{9.5\sqrt{R}} = \frac{2 \times 6^2}{2 \times 230} + \frac{80}{2.5\sqrt{230}} = 0.71 \text{ m}$$

$$\text{Total width of pavement} = 7 + 0.71 = 7.71 \text{ m}$$

HORIZONTAL TRANSITION CURVE :-

- T-curve is provided to change the alignment of pavement from straight to circular gradually.
- The T-curve has a radius which decreases from infinity at the Tangent Point to the designed Radius of circular curve.



$R_{\text{TC}, \text{max}}$ = Turning point

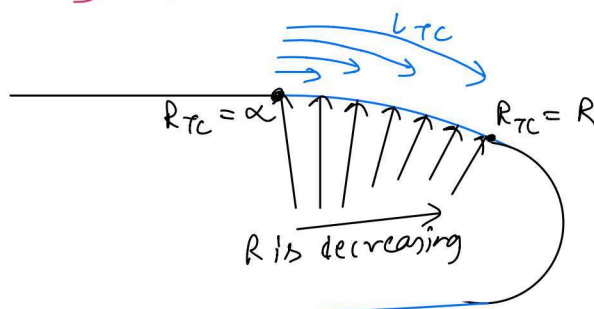
$R_{\text{TC}, \text{min}}$ = circular curve.

Objectives of providing the transition curve:-

- To introduce centrifugal force gradually over the vehicle.
- To reduce the sudden jerk
- To introduce super elevation and extra widening gradually.
- To improve the aesthetic appearance of road.

NOTE:-

1) For an ideal T-curve, the length of the curve should be inversely proportional to the radius.



→ More the length, less the radius.

$$L \propto \frac{1}{R} \quad LR = \text{const}$$

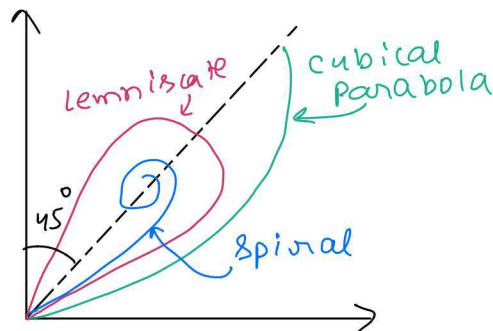
At TP, $R = \infty$, $L = 0$

At CC, $R = R_c$, $L = L_s$.

→ R is incr, centrifugal force is also increasing.

- Four transition curve to be provided following options are available

- (i) Spiral,
- (ii) Lemniscate.
- (iii) Cubical parabola.



- All the three curves follow same path upto deflection angle of 45° and are almost same upto 9° .
- In all these curves the radius decreases as length increases.
- But the rate of change of radius or rate of change of centrifugal force is not constant in case of lemniscate and cubical parabola. (Basic requirement)
- Hence SPIRAL CURVE is used as Transition curve as it fulfills the basic requirement of ideal transition curve.

- Length of the Transition curve is designed to fulfill three conditions :-

(i) Rate of change of centrifugal force :-

- length of the Transition curve is designed such that the rate of change of centrifugal force is low.

- Rate of change of acceleration (c)

$$c = \frac{\Delta a}{T} = \frac{a_f - a_i}{T}$$

$$c = \frac{\frac{v^2}{R_c} - 0}{L_s / v_d} = \frac{v_d^3}{L_s R_c} = \frac{(0.278 v)^3}{L_s R_s}$$

$$L_s = \frac{0.0215 v^3}{c R} \rightarrow \text{kmph (Design Speed)}$$

c = Rate of change of acceleration.

- The max allowable value of rate of change of acc "c" without producing discomfort to the vehicle/passenger is given by.

$$c = \frac{80}{75 + v} \text{ m/s}^3$$

$$c = \begin{matrix} \text{min} \\ \uparrow \\ (0.5 - 0.8) \text{ m/sec}^3 \end{matrix} \begin{matrix} \text{max.} \\ \uparrow \end{matrix}$$

(ii) Rate of introduction of super-elevation (N) :-

Here rate of introduction of super-elevation (N) : $N(H)$

l = Elevation

N = length of T-curve

As per IRC

1:150 → plain / Rolling terrain. T-curve

1:100 → for Built up area. Start

1:60 → for mountainous / steep terrain.

Hence length of TC = Nx

x = Elevation of outer edge.

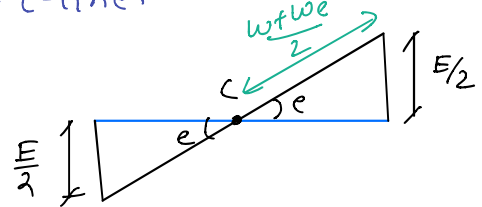
Case-(I) :- If rotation is done about C-line :-

$$\tan \theta = \frac{x}{\left(\frac{w+we}{2}\right)}$$

$$x = \frac{1}{2} e (w+we)$$

$$L_s = \frac{1}{2} e (w+we) N$$

$$\text{or, } L_s = \frac{NE}{2}$$



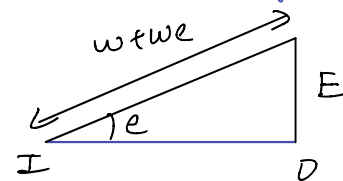
Case-(II) :- when rotation is done about inner edge

$$\tan \theta = \frac{x}{w+we}$$

$$x = e (w+we)$$

$$L_s = Nx = eN (w+we)$$

$$\text{or } L_s = NE$$



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(III) As per IRC :-

As per IRC, length of horizontal T-curve is given by following empirical relationship.

(a) for plain and rolling terrain :

$$L_s = \frac{2.7 v^2}{R} \quad \text{Kmph}$$

(b) for mountainous & steep :

$$L_s = \frac{v^2}{R}$$

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length of T-curve to be provided is max of three calculated from previous criteria.

- 8) A NH passing through rolling terrain in heavy rainfall area has a horizontal curve of radius 500m. Design the length of Transition curve for this highway.

sol For, NH in rolling terrain, $V_d = 80$ kmph

Radius = 500m

camber (heavy rainfall) = 2%.

- (i) Taking Rate of change of centrifugal force:-

$$L_{s1} = \frac{0.0215 V^3}{R C} \quad C = \frac{80}{75+V} = \frac{80}{75+80} = 0.516$$

$$L_{s1} = \frac{0.0215 \times 80^3}{500 \times 0.51} \quad 0.5 < C < 0.8$$

$$L_{s1} = 42.6 \text{ m.} \quad \text{Take } C = 0.51$$

- (ii) Taking Rate of change of superelevation criteria:-

For Rolling terrain, $N = 1$ in 150
 $N = 150$.

Design of super elevation,

$$e + f = \frac{V^2}{127R}$$

$$e = \frac{V^2}{225R} = \frac{80^2}{225 \times 500} = 0.056 \text{ (0.56\%)}$$

Hence provide $e = 0.056$

since $R > 300\text{m}$, $w_e = 0$.

$$\therefore L_{c2} = \frac{N e}{2} (W) \quad W = 2 \text{ lane} = 7 \text{ m. (Assume)}$$

$$= \frac{150}{2} \times 0.056 \times 7 = 29.4 \text{ m}$$

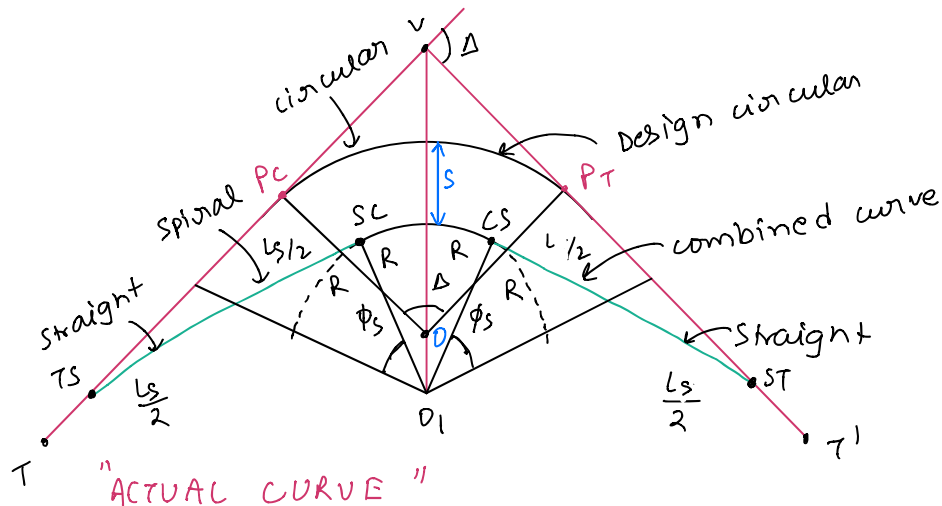
- (iii) IRC Recommendation.

$$\text{Rolling: } R = \frac{2.7 V^2}{R} = \frac{2.7 \times 6400}{500} = 34.56 \text{ m}$$

$$L_c = \max \{ L_{c1}, L_{c2}, L_{c3} \} = \max \{ 42.6 \text{ m}, 29.4 \text{ m}, 34.56 \text{ m} \}$$

$$= 42.6 \text{ m}$$

SETTING OF TRANSITION CURVE :-



L_s = length of T-curve.

PC = Point of curvature
PT = " " Tangency. SC = Transition curve to circular curve point.

CS = circular curve to Transition Curve
ST = Transition to straight point.

As shift "s" is small so we overlap "O" and "O'".
Shifting is necessary to as to fit circular curve with spiral.

(1) SHIFT / THROW

$$S = \frac{L_s^2}{24R}$$

L_s = length of T-curve.

(2) SPIRAL ANGLE (ϕ_s)

$$\phi_s = \frac{L_s/2}{R} \text{ (radian)}$$

$$\phi_s = \frac{180}{\pi} \times \frac{L_s}{2R} \text{ (degree)}$$

(3) Length of circular curve (L_{cc})

$$\Delta - 2\phi_s = \frac{2\pi R}{360} (\Delta - 2\phi_s)$$

$$L_{cc} = \frac{2\pi R}{360} (\Delta - 2\phi_s)$$

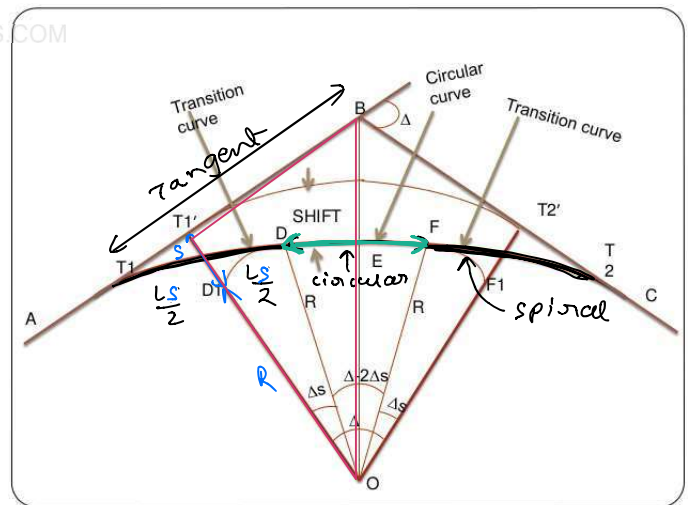
(4) Length of Tangent- (T_1B)

$$T_1T_1' = \frac{L_s}{2}$$

$$T_1'B = (R+s) \tan \frac{\Delta}{2}$$

$$L_s = T_1T_1' + T_1'B$$

$$L_s = \frac{L_s}{2} + (R+s) \tan \frac{\Delta}{2}$$



(5) CHAINAGE :-

- chainage of $T_1 = \text{chainage of B} - L_T$
Known.
- chainage of D = chainage of $T_1 + L_S$
- chainage of F = chainage of D + L_{CC}
- chainage of $T_2 = \text{chainage of F} + L_S$

Q) (a) On a national highway in rolling terrain two straight intercept at chainage 1000m with an angle of deflection of 40° . It is proposed to put a circular arc of 14 chain radius, with transition curve of 3 chains (20m length) at each end.

(b) Determine the chainages at the beginning of the first transition length circular curve and the second transition curve and show compounded data in neat sketch.

Sol Chainage at point B = 1000 m

$$\Delta = 40^\circ, R = 14 \times 20 = 280 \text{ m}$$

$$L_S = 3 \times 20 = 60 \text{ m}$$

$$S = \frac{L_S^2}{24R} = \frac{60^2}{24 \times 280} = 0.535 \text{ m.}$$

$$L_T = \frac{L_S}{2} + (R+S) \tan \frac{\Delta}{2} = \frac{60}{2} + (280 + 0.535) \tan \frac{40}{2}$$

$$L_T = 132.106 \text{ m.}$$

$$\text{Spiral Angle, } \phi_s = \frac{180}{2\pi R} L_S = \frac{180}{2\pi \times 280} \times 60 = 6.13^\circ$$

$$L_{CC} = \frac{2\pi R}{360} \times (\Delta - 2\phi) = \frac{2\pi \times 280}{360} (40 - 2 \times 6.13) = 135.4^\circ$$

$$\begin{aligned} \text{chainage of } T_1 &= \text{chainage of B} - L_T \\ &= 1000 - 132.106 = 867.89 \text{ m} \end{aligned}$$

$$\text{chainage of D} = 867.89 + L_S = 867.89 + 60 = 927.894 \text{ m}$$

$$\text{chainage of F} = \text{chainage of D} + L_{CC} = 927.894 + 135.4 = 1063.3$$

$$\text{" " } T_2 = \text{" " F} + L_S = 1063.37 + 60 = 1123 \text{ m}$$

Draw the diagram as previous.

Jan 12 # SETBACK DISTANCE :-

- It is the clearance distance required from the centerline of pavement / road to the obstruction in order to maintain the adequate sight distance to the curve.
- The set back distance or clearance which is required from the centreline of horizontal curve depends upon
 - (a) required sight distance.
 - (b) Radius of H-curve.
 - (c) length of the curve. (L_c).

case (i) :- when $L_c > SD$:- SD can be anything.

(a) For narrow roads such as single lane road :-

$O \rightarrow$ obstruction.
 $AB \rightarrow$ sight line.

$OD = m =$ setback distance.

$$m = CD - CO$$

$$m = R - CO$$

In $\triangle ACO$

$$\cos \frac{\alpha}{2} = \frac{CO}{R} \quad CO = R \cos \frac{\alpha}{2}$$

$$m = R - R \cos \frac{\alpha}{2}$$

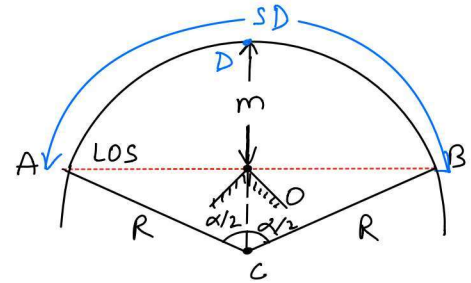
$$m = R (1 - \cos \frac{\alpha}{2})$$

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$$\widehat{AD} = R \frac{\alpha}{2}$$

$$\frac{\alpha}{2} = \frac{(SD)}{2 \cdot R} \text{ (radians)}$$

$$\alpha = \frac{(SD)}{R} \times \frac{180}{\pi} \text{ (degree)}$$



NOTE:- SD is calculated along the centre line of road in single lane road

$m \rightarrow$ setback distance measured from centre-line.

(b) wide Roads / two or more lane Roads :-

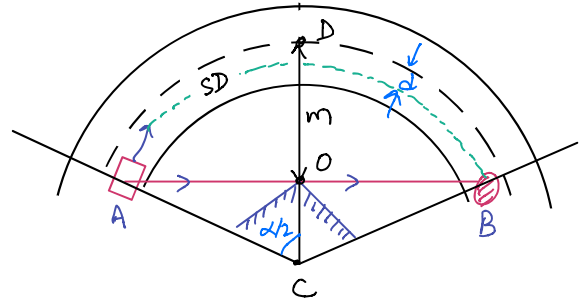
- Here the stopping distance is measured along the centre line of the inner lane and the setback distance is measured from the centre of the road.
- Let "d" be the distance b/w the centre line of the road and centreline of the inside lane.

$$CB = R - d.$$

$$d = \frac{w + w_e}{4}$$

$$\alpha = \frac{SD}{(R-d)} \times \frac{180}{\pi} \cdot (\text{degree})$$

$$m = R(1 - \cos \alpha/2) + d \cos \alpha/2$$



(d) Inner edge of u $\quad u$ $\quad u$ $\quad = m-2d$

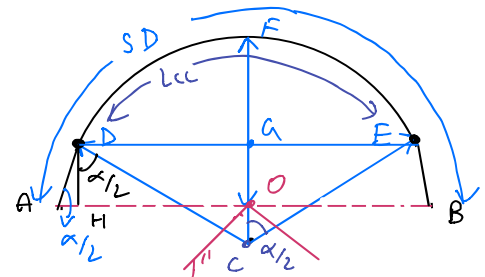
(a) Four single lane Road :-

$$m = F_0 = F_u + u_0$$

In A GCD

$$\cos \frac{\alpha}{2} = \frac{4c}{R}$$

$$h_c = R \cos \alpha/2$$

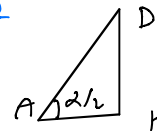


$$AD = \frac{SD - ICC}{2}$$

$$\sin \frac{\alpha}{2} = \frac{DH}{AD}$$

$$DM = GO = \left(\frac{SD - l_{cc}}{2} \right) \sin \alpha/2$$

$$m = R(1 - \cos \alpha/2) + \left(\frac{SD - L_{CC}}{2} \right) \sin \frac{\alpha}{2}$$



$$\frac{\alpha}{2} = \frac{L_{cc}}{2R}$$

$$\alpha = \frac{l_{cc}}{R} \text{ (radian)}$$

$$\alpha = \frac{I_{cc}}{I} \times \frac{180}{\pi} \text{ (degree)}$$

(b) Four Multi lane Road -

$$\frac{\alpha}{2} = \frac{L_{cc}}{2(R-d)} \text{ (radian)}, = \frac{180}{\pi} \frac{L_{cc}}{2(R-d)} \text{ (deg)}$$

$$m = R(1 - \cos \alpha/2) + d \cos \alpha/2 + \left(\frac{SD - L_{cc}}{2} \right) \sin \alpha/2$$

NOTE:- Setback distance from centre line of inner lane = $m - d$.
Setback distance from centre line of outer lane = $m + d$
and so on. (For two lane road)

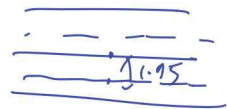
- Q) A two lane highway has a horizontal curve of Radius 200m & total length of the curve is 240m. The distance between the centre line of the highway & the centre of inner lane is 1.95m. If the desired sight distance is 340m, Determine the set back distance required.

Sol/

$$R = 200\text{m} \quad L_{cc} = 240\text{m} \quad SD = 340\text{m}$$

($L_{cc} < SD$)

$$m = R(1 - \cos \alpha/2) + d \cos \alpha/2 + \left(\frac{SD - L_{cc}}{2} \right) \sin \alpha/2$$



$$\alpha = \frac{L_{cc}}{(R-d)} \times \frac{180}{\pi} = \frac{240}{(200-1.95)} \times \frac{180}{\pi} = 69.43^\circ$$

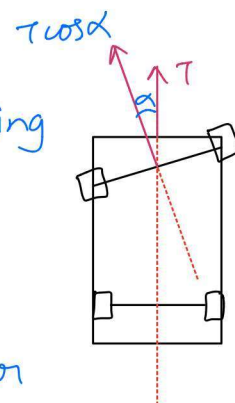
$$m = 200(1 - \cos 34.71) + 1.95 \cos 34.71 + \left(\frac{340 - 240}{2} \right) \sin 34.71$$

$$m = 200(1 - 0.822) + 1.95 \times 0.822 + 50 \times 0.569$$

$$= 65.65 \rightarrow \text{about centre of two lane}$$

Jan 13 # CURVE RESISTANCE :-

- The automobiles are steered by turning the front wheels but the rear wheel donot turn.
- when a vehicle is driven by gear when on a horizontal curve, the direction of rotation of gear & front are different.



- If tractive force "T" is provided to the rear wheel in direction normal to its axis, then the tractive force available for the front wheel is $T \cos \alpha$ here α is angle of turn.

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- Hence loss of tractive force due to turning of vehicle on H-curve is $= T - T \cos \alpha$ is termed as curve resistance.

8) A state highway passing through rolling terrain has a horizontal curve of radius equal to rolling min radius.

(i) Design all the geometric features of this horizontal curve.

(ii) Specify the min set back distance from the centre line of two lane highway on the inner side of the curve upto which the buildings etc. obstructing the vision should not be constructed so that intermediate sight distance is available throughout the circular curve. Assume the length of circular curve is greater than the SD.

Sol For State Highway in rolling terrain.

$$RMR = 230 \text{ m.}, V_d = 80 \text{ kmph.}$$

(i)

(a) Superelevation

$$e = \frac{V^2}{225R} = \frac{80^2}{225 \times 230} = 0.123 \quad (> 7\%)$$

Restrict $e = 0.07$ and find f .

$$f = \frac{80^2}{127 \times 230} - 0.07 = 0.149 \approx 0.15$$

Hence provide $e = 0.07$ and $f = 0.15$

(b) width of pavement

$$W_e = \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}}$$

Assuming 2-lane road and $l = 6.1 \text{ m}$

$$W_e = \frac{2 \times (6.1)^2}{2 \times 230} + \frac{80}{9.5\sqrt{230}} = 0.71 \text{ m}$$

$$\text{Total width} = 7 + 0.71 = \underline{7.71 \text{ m}}$$

(c) length of transition curve

(i) Rate of superelevation criteria.

$$L_c = \frac{N_e}{2} (W + W_e) \quad \left\{ \begin{array}{l} \text{Rotation about inner} \\ \text{edge} \end{array} \right.$$

$N = 150$ (1:150) for rolling terrain.

$$L_{s1} = \frac{150 \times 0.07}{2} \times 7.71 = 40.47 \text{ m}$$

(ii) Rate of centrifugal force criteria

$$L_{s2} = \frac{0.0215 V^3}{CR} \quad C = \frac{80}{75+V} = \frac{80}{75+80} = 0.51$$

$$L_{s2} = \frac{0.0215 \times 80^3}{0.51 \times 230} = 93.84 \text{ m}$$

(iii) As per IRE

$$L_{s3} = \frac{2.5 V^2}{R} = \frac{2.5 \times 80^2}{230} = 69.56 \text{ m}$$

Hence $L_s = \max \{L_{s1}, L_{s2}, L_{s3}\} = 93.84 \text{ m}$.

(ii) Set back distance

$$L_{cc} > SD$$

$$m = R(1 - \cos \alpha/2) + d \cos \alpha/2$$

$$\frac{\alpha}{2} = \frac{180 SD}{\pi^2 (R-d)} \quad \text{ISD} = 2 \times SSD$$

$$\alpha = \frac{180 \times 255}{\pi (230 - 7.71)} = 2 \left[0.278 V + \frac{V^2}{254.5} \right]$$

$$= 2 \times \left[0.278 \times 80 \times 2.5 + \frac{80^2}{254 \times 0.35} \right]$$

$$\frac{\alpha}{2} = 32.03 \quad \text{ISD} = 255$$

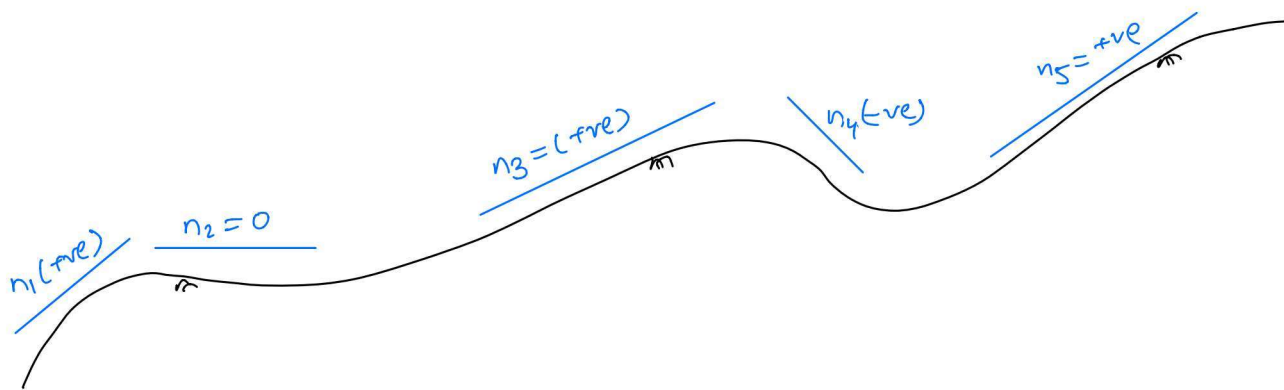
$$m = 230 (1 - \cos 32.03) + \frac{7.71}{2} \cos 32.03$$

$$= 30.19 \text{ m}$$

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VERTICAL ALIGNMENT

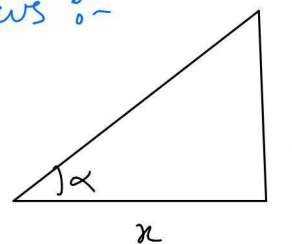


- The natural ground may be level at some places and may have slope of different magnitude at other locations.
- Generally highway is aligned to follow the natural topography keeping in view the drainage and other design consideration.
- The vertical alignment of the highway affects
 - (i) acceleration & deceleration.
 - (ii) Operational cost of vehicle.
 - (iii) Speed
 - (iv) sight distance
 - (v) comfort while driving.
- The vehicle alignment consist of two elements
 - (i) GRADIENT
 - (ii) VERTICAL CURVE

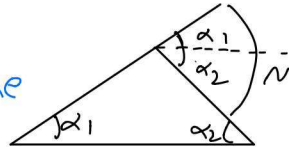
(I) GRADIENT :-

- It is defined as change in elevation / ht of road along the road or may be defined as rise or fall along the length of road wrt horizontal.
- It can be represented as follows :-

- (a) 1 in x
- (b) $\frac{1}{x} \times 100 \%$
- (c) $\tan \alpha$



Note :- The angle which measures change in direction at the intersection of two grades is called as deviation angle.



$$N = \alpha_1 + \alpha_2$$

$$\text{or, } N = \alpha_1 - (-\alpha_2)$$

- Types of Gradient :-

Gradients are divided into following :-

(i) Rolling gradient / DESIGN GRADIENT :-

- It is the maximum gradient with which the designer attempts to design the vertical profile of road.
- Gradients upto rolling gradient are adopted as a normal course in design.
- However flatter gradients may be preferred wherever it can be provided.

(ii) LIMITING GRADIENT :-

- wherever topography of a place forces to adopt steeper gradient than rolling gradient, it can be extended upto limiting gradient or else enormous cost of construction would be observed.
- However the length of road on which this gradient is provided is limited.

(iii) EXCEPTIONAL GRADIENT :-

- In some extreme situations, it may be unavoidable to provide still steeper gradients than limiting gradient for short stretches of road, in such cases exceptional grad. can be provided but upto 100 m. at a stretch

As per IRC values of diff gradient are as follows :-

TYPE OF TERRAIN	RULING	LIMITING	EXCEPTIONAL
- PLAIN OR ROLLING	1 in 30	1 in 20	1 in 15
- MOUNTAINOUS & STEEP terrain having RL more than 3000m above MSL	1 in 20	1 in 16.7	1 in 14.3
- STEEP terrain up to RL 3000 m above MSL	1 in 16.7	1 in 14.3	1 in 12.5

NOTE:- Above 3000m, oxygen less so difficult in combustion of engine hence optimum power is not generated.

(iv) MINIMUM GRADIENT:-

- The min gradient to be provided from drainage point of view in order to allow gravity flow of water (Rain) inside the drainage system.

- As per IRC min gradient are as follows.

(i) CONCRETE DRAIN - $1/500$ (0.2%)

(ii) KUTCHA/SOIL DRAIN - $1/200$ (0.5%)

- Depending upon type of soil it can be taken up to $\frac{1}{100}$ (1%) ← highly permeable soil

Magnitude :- $E_G > L_G > R_G/D_G > M_G$

Jan 14 NOTE:- GRADE COMPENSATION ON HORIZONTAL CURVE:-

- At horizontal curve due to the turning angle of vehicles with more wheel drive curve resistance of $(T - T \cos \alpha)$ is developed.

- When there is horizontal curve in addition to the gradient there will be increased resistance due to both horizontal curve (curve resistance) and gradient (grade resistance).

Total Resistance = grade + curve

- Hence when sharp horizontal curve is introduced on a road which has already the max permissible gradient (Ruling), the reduction in the gradient is to be carried out.
- This reduction in gradient at H-curve is termed as grade compensation.

$$GC(\%) = \frac{30+R}{R} \quad R = \text{Radius of H-curve (m)}$$

- The maximum value of GC is limited to $\frac{75}{R}$ as per IRC.

Thus, $GC = \min \left\{ \begin{array}{l} \frac{30+R}{R} \\ \frac{75}{R} \end{array} \right\}$ (*)

- GC is not necessary for gradient flatter than 4%.
hence it is not being provided beyond (less than) 4%

Q) what will be the compensated gradient?

- (a) If Ruling gradient is 6% on a H-curve of radius 60m.
(b) If Ruling gradient is 5% on a H-curve of radius 60m.

Also report GC in both the cases.

Sol (a) $R = 60 \text{ m}$

$$GC(\%) = \min \left\{ \begin{array}{l} \frac{30+R}{R} = \frac{30+60}{60} = 1.5\% \\ \frac{75}{R} = \frac{75}{60} = 1.25\% \end{array} \right.$$

$$GC(\%) = 1.25\%$$

$$\text{compensated grade} = 6 - 1.25 = 4.75\% \quad (> 4\%)$$

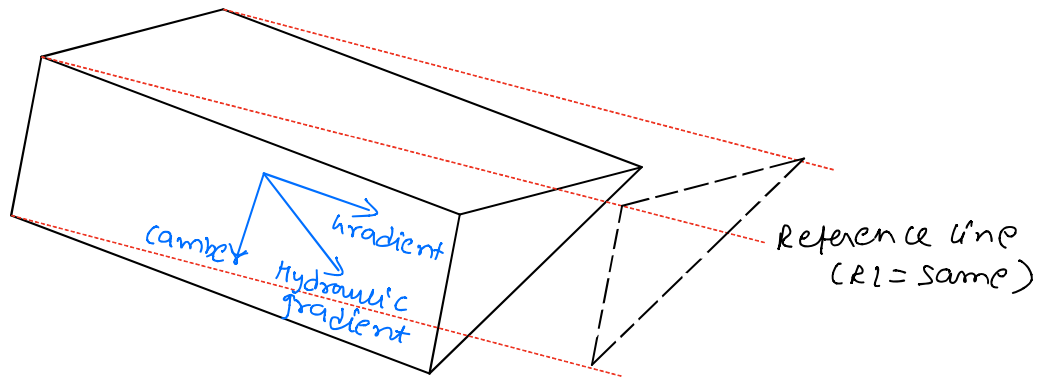
(b) $R = 60 \quad GC(\%) = 1.25\%$

$$\text{compensated grade} = 5 - 1.25 = 3.75\% \quad (< 4\%)$$

$$\text{hence compensated gradient} = 4\%$$

$$\therefore GC = 5 - 4 = 1\%$$

NOTE :-



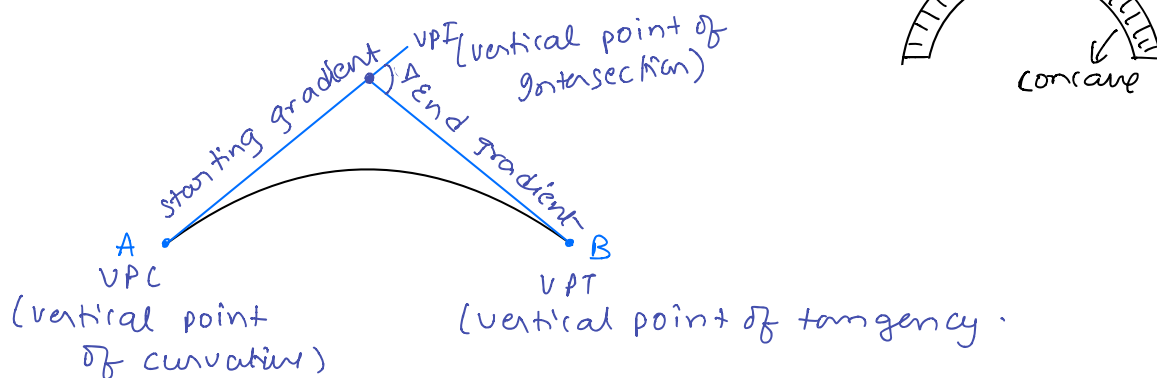
- when camber and gradient is provided together to the road, water flows in direction resultant to the camber known as "HYDRAULIC GRADIENT"

(II) VERTICAL CURVE :-

- Due to change in vertical alignment of road, it is necessary to introduce vertical curve at the intersection of diff. grades to smoothen out the ride.
- These curve are broadly classified into two
 - (i) SUMMIT / CREST CURVE with CONVEXITY upward
 - (ii) VALLEY CURVE / SAG CURVE with CONCAVITY upward

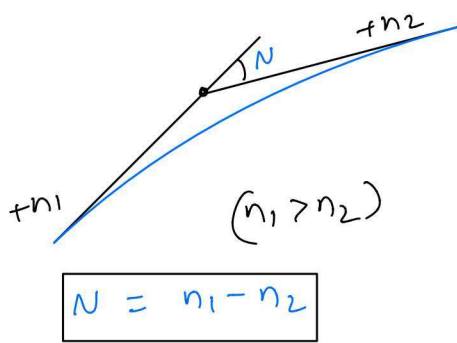
(I) SUMMIT CURVE :-

- These are vertical curves with convexity upward & concavity downward.

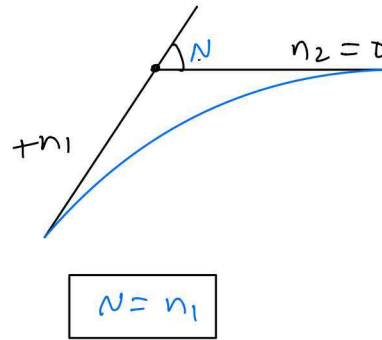


- Summit curve is formed by two gradient. (starting & end gradient) as follows.

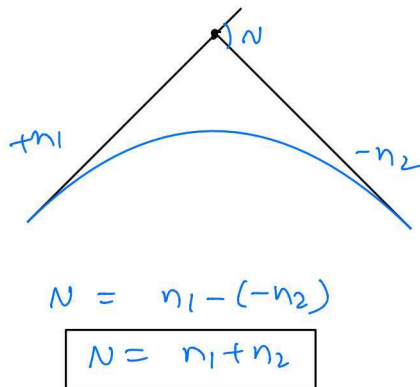
(a) (+ve) to (+ve)



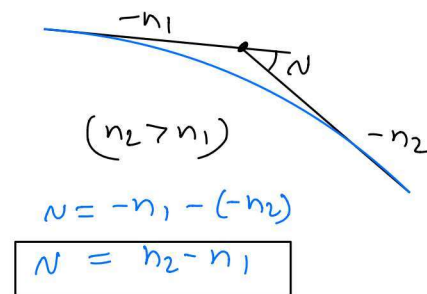
(b) (+ve) to flat



(c) +ve to -ve



(d) -ve to -ve



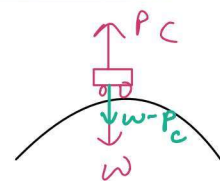
- Significant points regarding Summit curve:-

(i) vertical point of Intersection (VPI) always lies above the curve.

(ii) Summit curve are designed only for SD criteria.
 $mn \rightarrow SSD \rightarrow$ must be provided
 desirable $\rightarrow$ OSD
 if OSD not possible go for $ISD = 2 \times SSD$.

(iii) when a fast moving vehicle travels along a summit curve centrifugal force acts upwards against gravity hence effective wt of vehicle is reduced. There by pressure on tyre and suspension are relieved.
 Hence there is "NO PROBLEM OF DISCOMFORT".

(iv) Since design of summit curve is governed by stopping distance or sight distance criteria, CIRCULAR curve is ideal for this, as the SD in this case remains constant. Hence it is available throughout the curve.



- (v) The deviation angles in vertical curve are very small, hence a simple "PARABOLA" or "SQUARE PARABOLA" is nearly analogous to CIRCULAR arc, also a parabola is easily to manipulate mathematically for computing. Moreover it offers good riding comfort. Hence being provided practically

ANALYSIS OF CURVE

- This curve is non-symmetrical as $n_1 \neq n_2$.

(i) GENERAL EQUATION

$$y' = ax^2 + bx + c$$

@ Pt A :- $x=0, y'=0$

$$c=0$$

$$\frac{dy'}{dx} = 2ax + b \quad (\text{gradient/slope})$$

@ Pt A, $x=0, \frac{dy'}{dx} = n_1$

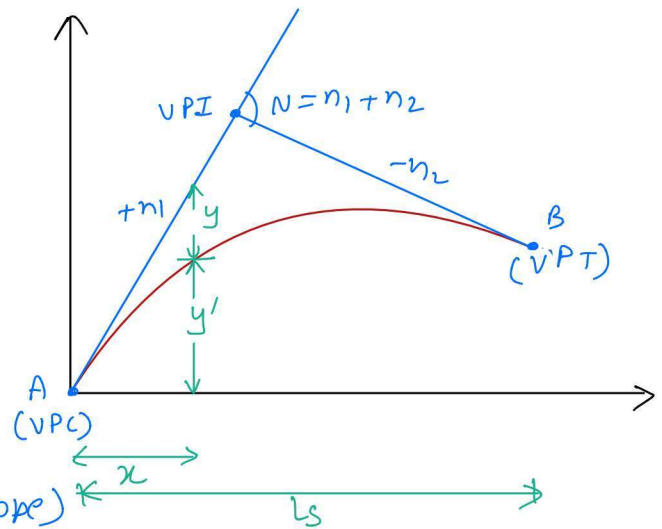
$$n_1 = b$$

@ Pt B, $x=L_s, \frac{dy'}{dx} = -n_2$

$$-n_2 = 2aL_s + n_1$$

$$a = \frac{-(n_2 + n_1)}{2L_s}$$

$$a = -\frac{N}{2L_s}$$



$$y' = ax^2 + bx + c$$

$$y' = \frac{-N}{2L_s} x^2 + n_1 x$$

General Eqn

NOTE:- here general eqn means normal x-y axis parameter here x and y'.

(ii) Position of summit / crest / highest point of curve from VPC

maximize y' , i.e. $\frac{dy'}{dx} = 0$.

$$\frac{dy'}{dx} = 2ax + b = 0$$

$$x = -\frac{b}{2a} = \frac{-n_1}{2 \left(-\frac{N}{2L_s} \right)} = \frac{n_1 L_s}{N}$$

$$x = \frac{n_1 L_s}{N}$$

(iii) APEX EQUATION

Note:- Here apex means parameter above the road or ground i.e. (x and y)

At A, $\tan \theta = n_1$
 $\frac{y+y'}{x} = n_1$
 $y = n_1 x - y'$

→ $y = n_1 x + \frac{N}{2L_s} x^2 - n_1 x$

$y = \frac{N}{2L_s} x^2$

- Q) An ascending gradient of 2% meets a descending gradient of 3% to form a summit curve. If the length of the curve is 150 m, then find out the summit point from the starting of ascending gradient. Also frame the eqn of summit curve.

Sol = $n_1 = 0.02$ $N = 0.05$
 $n_2 = -0.03$
 $L_s = 150 \text{ m}$

$$x = \frac{n_1 L_s}{N} = \frac{0.02 \times 150}{0.05} = 60 \text{ m from start}$$

$$y' = -\frac{N}{2L_s} x^2 + n_1 x$$

$$y' = -\frac{0.05}{2 \times 150} x^2 + 0.02x \quad \checkmark \quad \text{or} \quad y' = -\frac{1}{60} \% x^2 + 2 \% x$$

Ex 15 (iv) Position of VPI from VPC

In $\triangle AVE$

$$\tan \theta_1 = n_1 = \frac{VE}{x}$$

$$VE = n_1 x \quad \text{--- (i)}$$

In $\triangle VBD$

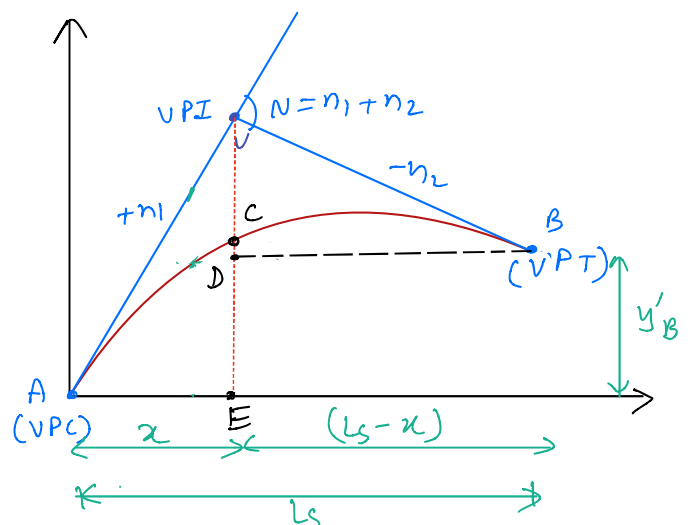
$$\tan \theta_2 = n_2 = \frac{VD}{(L_s - x)}$$

$$VD = n_2 (L_s - x) \quad \text{--- (ii)}$$

$$\text{and, } y'_B = -\frac{N}{2L_s} L_s^2 + n_1 L_s = -\frac{N}{2} L_s + n_1 L_s$$

$$\text{Now, } y'_B = VE - VD$$

$$n_1 x - n_2 L_s - n_2 x = -\frac{N}{2} L_s + n_1 L_s$$



$$x = \frac{L_s}{2}$$

- The above relation is valid for both symmetrical & unsymmetrical curve.

NOTE (I):- when curve is symmetrical :-

- Vertical line passing through VPI will bisect the length of curve (L_s) in 2- equal half.
- This line will always pass through summit point of the curve.
- In this case, RL of starting pt. and end point will be same.

NOTE (II):- when curve is unsymmetrical:-

- Vertical line passing thru VPI will bisect the length of curve in two equal halves.
- This line will never pass through summit point of the curve.
- In this case RL of starting and end point are different and the curve will be lifted.

(v) RL calculation :-

If RL of VPL (Pt. A) is known.

(a) RL of point just below the VPI on summit curve (Pt. C)

$$\begin{aligned} RL_C &= RL_A + y' \Big|_{x=\frac{L_s}{2}} \\ &= RL_A + \left(-\frac{N}{2L_s}\right) \frac{L_s^2}{4} + n_1 \frac{L_s}{2} \end{aligned}$$

$$RL_C = (RL)_A + \left[n_1 \frac{L_s}{2} - \frac{N L_s}{8} \right]$$

(a) RL of End point. (Pt. B)

$$\begin{aligned} RL_B &= RL_A + y' \Big|_{x=L_s} \\ &= RL_A + \left(-\frac{N}{2L_s}\right) L_s^2 + n_1 L_s \end{aligned}$$

$$RL_D = RL_B = RL_A + \left[n_1 L_s - \frac{N L_s}{2} \right]$$

Expanding $N = n_1 + n_2$

$$\begin{aligned} RLB &= RLA + n_1 L_s - \frac{n_1 L_s}{2} - \frac{n_2 L_s}{2} \\ &= RLA + \frac{n_1 L_s}{2} - \frac{n_2 L_s}{2} = RLA + L_s \left(\frac{n_1 - n_2}{2} \right) \end{aligned}$$

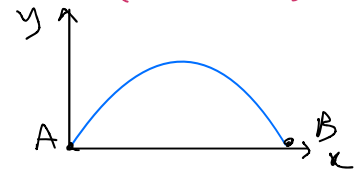
From above eqn:-

(i) For symmetric curve :- $n_1 = n_2$ i.e. $RLB = RLA$
(note 1 - 3rd point)

(ii) For unsymmetrical curve: $n_1 \neq n_2$

(a) $n_1 > n_2$

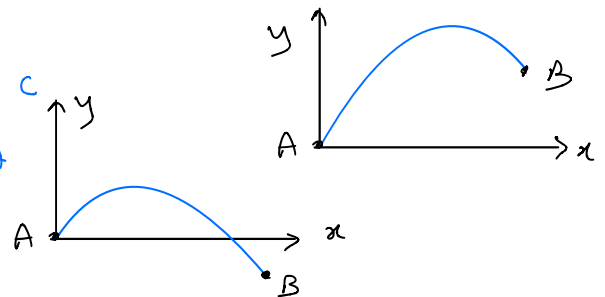
$$RLB = RLA + C \quad \text{i.e. } RLB > RLA$$



(a) $n_2 > n_1$

$$RLB = RLA - C$$

i.e. $RLB < RLA$



LENGTH OF SUMMIT CURVE :-

- while designing the length of parabolic summit curve it is necessary to consider SSD & OSD respectively.

L_s = length of summit curve.

(case I) :- length of summit curve for SSD

(a) $L_s > SSD$ (as in diagram)

$$L = \frac{NS^2}{(\sqrt{2H} + \sqrt{2h})^2}$$

→ From Apex
eqn

S = sight distance

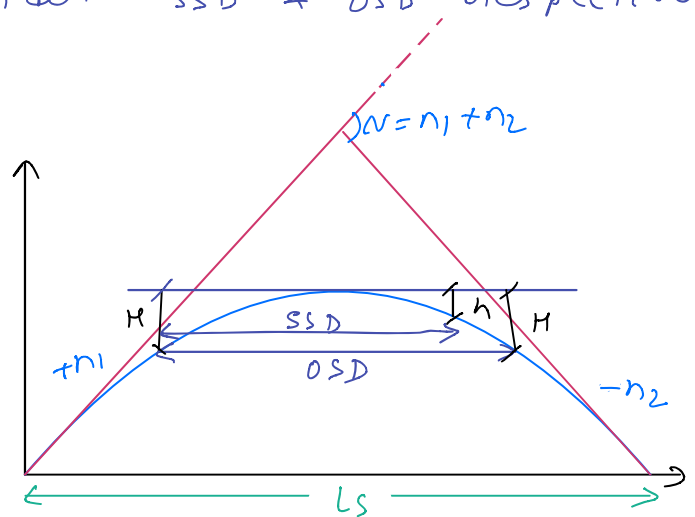
$$@ x = x_1, y = H$$

$$@ x = x_2, y = h$$

$$SSD = x_2 - x_1$$

As per IRC, $H = 1.2 \text{ m}$, $h = 0.15 \text{ m}$

$$L = \frac{NS^2}{4.4}$$



h = height of object
 H = " " drivers

(b) $L_s < SSD$

$$L = 2S - \frac{(\sqrt{2H} + \sqrt{2h})^2}{N}$$

As per IRC,

$$L = 2S - \frac{4.4}{N}$$

NOTE:- While calculating the length of summit curve first assume case (A) $L > SSD$. If it fails then go for case (B).

case (II):- length of Summit curve (L) from OSD criteria

(a) $L > OSD$

$$L_s = \frac{NS^2}{(\sqrt{2H} + \sqrt{2h})^2} = \frac{NS^2}{8H^2}$$

As per IRC $H = 1.2m$

$$L_s = \frac{NS^2}{9.6}$$

* Don't Remember by heart. Remember gen. eg. as in que h.t. of driver or object can be changed.

(b) $L < OSD$

$$L_s = 2S - \frac{(\sqrt{2H} + \sqrt{2h})^2}{N} = 2S - \frac{8H}{N}$$

As per IRC,

$$L = 2S - \frac{9.6}{N}$$

NOTE:- 1) when deviation angle are strong the length of summit curve generally works out less than sight distance.

2) In very small deviation angle (α), the length of summit curve works out to be negative, indicating that there is no problem of sight restriction at the summit curve.

3) For calculating SD ($SSD + OSD$), the effect of gradient is not to be considered.

4) The minimum radius of the curve " R " is given by

$$R = \frac{L}{N}$$

5) The minimum length of vertical curve for diff speeds and for max. grade change which don't require vertical curve as per IRC are

given as follows.

Design Speed (kmph)	Max grade change (%) not requiring a vertical curve	Min. length of vertical curve
35	1.5	15
40	1.2	20
50	1.0	30
65	0.8	40
80	0.6	50
100	0.5	60

Q7) Design summit curve for a NH for SSD of 180 m at the junction of rising gradient $\frac{1}{50}$ and falling gradient of $\frac{1}{30}$. Determine the RL of a point immediately below the intersection pt. of tangent line. Also find RL of highest point on the curve. Assume RL at starting of curve of as 100 m.

Sol

$$SSD = 180 \text{ m}$$

$$n_1 = \frac{1}{50}, \quad n_2 = -\frac{1}{30}$$

$$N = \frac{1}{50} + \frac{1}{30} = \frac{3+5}{150} = \frac{8}{150}$$

length of summit curve

$$L > SSD \quad \therefore \quad L = \frac{NS^2}{4.4} = \frac{\frac{8}{150} \times 180^2}{4.4} = 392.92 \text{ m}$$

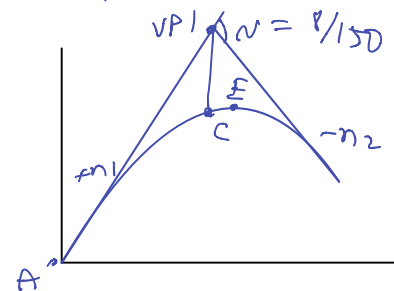
$$RL_C = RL_A + y' \Big|_x = \frac{Ls}{2}$$

$$= 100 + \left(\frac{-N}{2Ls} \right) \frac{Ls^2}{4} + n_1 \frac{Ls}{2}$$

$$= 100 - \frac{N}{8} Ls + n_1 \frac{Ls}{2}$$

$$= 100 - \frac{8}{150 \times 8} \times 393 + \frac{1}{50} \times \frac{393}{2}$$

$$= 101.31 \text{ m}$$



$$RL_E = RL_A + y' \Big|_x = \frac{n_1 Ls}{N} \quad x = \frac{1}{50} \times$$

$$= 100 + \left(\frac{-N}{2Ls} \right) \times \frac{n_1^2 Ls^2}{N^2} + n_1 \frac{n_1 Ls}{N}$$

$$= 100 - \frac{n_1^2 Ls}{2N} + \frac{n_1^2 Ls}{N} = 100 + \frac{n_1^2 Ls}{2N}$$

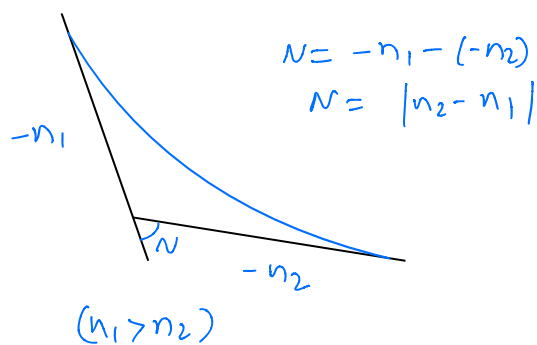
$$= 100 + \frac{(1/50)^2 \times 393}{2 \times 8/150} = 101.47 \text{ m}$$

Jan 16 (II) VALLEY CURVE :-

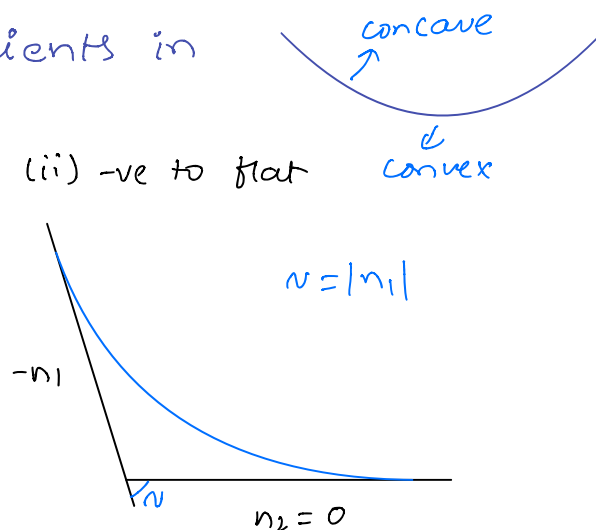
- This are the vertical curve with concavity upward and convexity downward.

- It is also formed by two gradients in following ways.

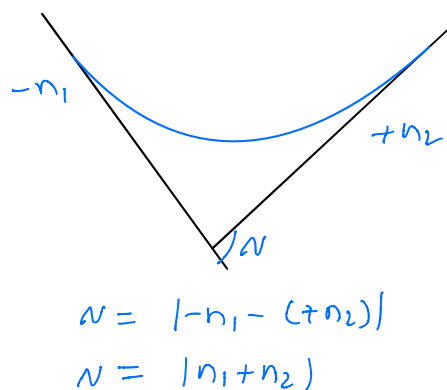
(i) -ve to -ve



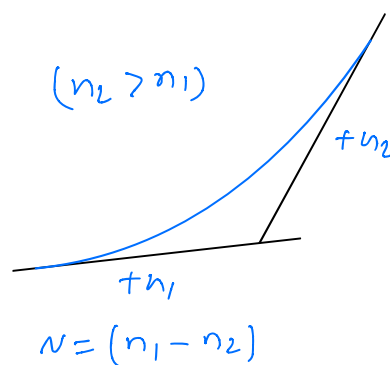
(ii) -ve to flat



(iii) -ve to +ve



(iv) +ve to +ve



- Some imp points of reference for valley curve :-

- VPI will always lie below the curve.
- As fast moving vehicle negotiate valley curve the centrifugal force developed acts downward in addition to the self wt, thus additional pressure on the suspension system is created. hence discomfort to the passenger due to impact is created.
- In order to avoid discomfort due to impact caused by centrifugal force acting downward, the rate of change of centrifugal acc. should be induced gradually along the valley curve.

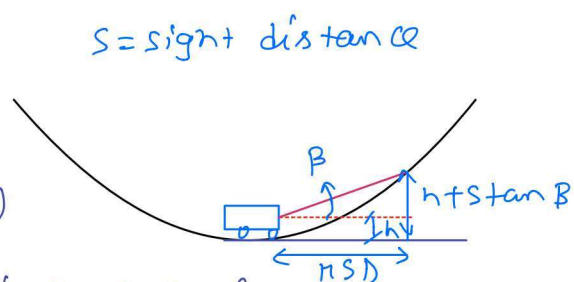
For this transition curve is to be provided, infact valley curve is formed by joining two T-curve.

NOTE:- "CUBIC PARABOLA" is generally preferred in valley curve.

- During night driving the visibility ahead is dependent on head light of vehicles, when the road lighting is not adequate.
- There is restriction of SD at valley curve as the head light get intercepted by the ascending portion of the curve.
- hence in this case SSD must be equal to HSD (head light sight distance). ($HSD_{min} = 0.75m$)
- However there is no problem of overtaking SD during night time as other vehicle with head light can be seen from considerable distance.

→ In India min ht of headlight is considered to be 0.75m

$h = 0.75m$
 β = Beam angle (generally 1°)



→ valley curve are designed, taking following considerations into account.

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- (i) comfort to passenger
- (ii) SSD at night. (HSD)
- (iii) Drainage in the valley curve at lower most pt.
- (iv) Aesthetic consideration.

(a) Position of lowest point of valley curve from starting of the curve (VPC) is given by:-

$$x = \left(\frac{n_1}{2N} \right)^{1/2} L_v \quad \text{where } L_v = \text{length of valley curve.}$$

(b) length of valley curve:-

- The valley curve and its length are designed as transition curve to fulfill two criteria

- (a) Allowable rate of change of acceleration.
- (b) Required HSD at night.

- The max of two values is adopted.

- Generally second criteria i.e HSD is higher, hence governs the design of the valley curve.

(a) length of Transition from compound condition

L_s = length of T-curve.

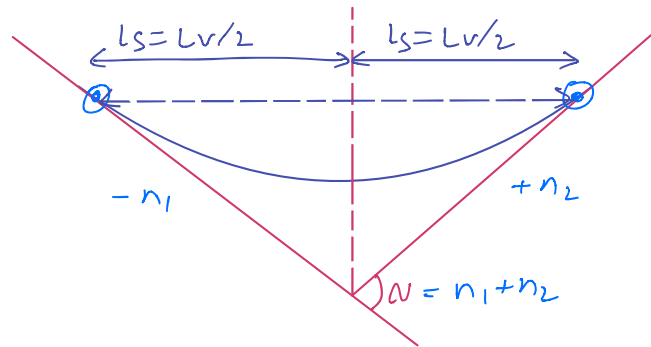
$$L_s = \frac{v^3}{cR}$$

$$R = \frac{L_s}{N} = \frac{Lv}{2N}$$

$$L_s = \frac{v^3}{cLv} (2N)$$

$$Lv^2 = \frac{4Nv^3}{c}$$

$$Lv = 2 \left(\frac{Nv^3}{c} \right)^{1/4}$$



$$0.5 < c = \frac{80}{75+v} < 0.8 \text{ m/s}^3$$

c is taken to be 0.6 m/s^3
and design speed in kmph

$$Lv = 2 \left(\frac{N (0.278v)^3}{0.6} \right)^{1/4}$$

$$Lv = 0.378 \sqrt{Nv^3} \text{ (m)}$$

→ (kmph)

(b) length of valley curve for MSD.

case (i) $L > \text{MSD} = \text{SSD}$:-

For sake of simplicity we assume the valley curve to be simple parabola

$$y = \left(\frac{N}{2Lv} \right) x^2$$

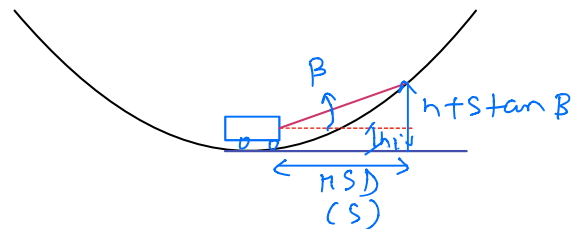
when $x = s$, $y = h_1 + s \tan \beta$

$$h_1 + s \tan \beta = \left(\frac{N}{2Lv} \right) s^2$$

$$Lv = \frac{Ns^2}{2(h_1 + s \tan \beta)}$$

As per IRC, $h_1 = 0.75 \text{ m}$, $\beta = 1^\circ$

$$Lv = \frac{Ns^2}{1.5 + 0.035s}$$



case (ii): $L < SSD$:-

$$h_1 + S \tan \beta = R \times N$$

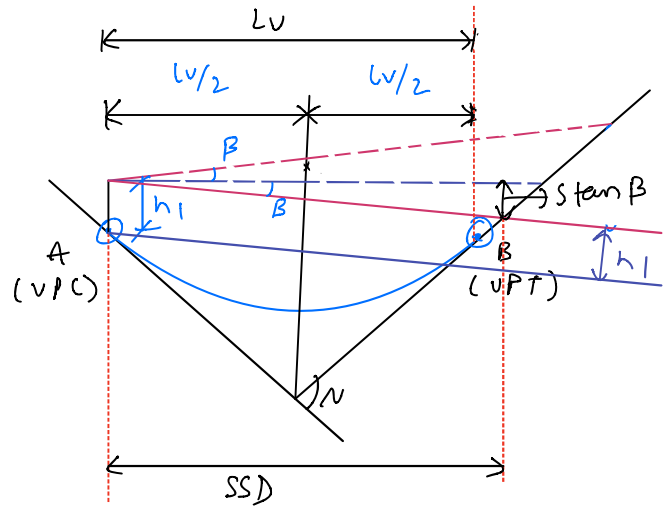
$$h_1 + S \tan \beta = \left(S - \frac{L_v}{2} \right) N$$

$$2h_1 + 2S \tan \beta = 2SN - L_v N$$

$$L_v = \frac{2S - 2(h_1 + S \tan \beta)}{N}$$

As per IRC, $h_1 = 0.75$, $\beta = 1^\circ$

$$L_v = \frac{2S - 1.5 + 0.0375 S}{N}$$



- Q) A valley curve is formed by a descending gradient of $1/25$ and an ascending gradient of $1/30$. Design the length of valley curve to fulfill both comfort condition and MSD condition, consider design speed 80 kmph and $f = 0.25$

Sol/ = $n_1 = -1/25$ $n_2 = +1/30$ $N = \frac{1}{25} + \frac{1}{30} = \frac{50}{750} = \frac{1}{15}$

(a) comfort condition

$$L_v = 0.378 \sqrt{N V^3} = 0.378 \sqrt{\frac{1}{15} \times 80^3} = 69.83 \text{ m}$$

(b) MSD condition

let $L > SSD$

$$L_v = \frac{NS^2}{1.5 + 0.035S}$$

$$S = 0.278 V_d + 0.1 + \frac{V_d^2}{254 f} = 0.278 \times 80 \times 2.5 + \frac{80^2}{254 \times 0.25}$$

$$S = 127.6 \text{ m}$$

$$\therefore L_v = \frac{\frac{1}{15} \times (127.6)^2}{1.5 + 0.035 \times 127.6} = 181.91$$

Hence assumpt. was true

provide $L = 182 \text{ m}$.

Jan 17

Traffic Engineering

- TE deals with improvement of traffic performance on road network through systematic traffic studies, analysis, planning, administration and engineering application.
- Here traffic performance represent : speed, safety, cost and efficiency of traffic movement
- Traffic engineering deals with following characteristics
 - (a) TRAFFIC CHARACTERISTICS.
 - (b) TRAFFIC STUDIES.
 - (c) TRAFFIC PLANNING & STUDIES.
 - (d) GEOMETRIC DESIGN.
 - (e) REGULATION, CONTROL & OPERATION.
 - (f) ROAD SAFETY ASPECTS.
 - (g) TRAFFIC MANAGEMENT & ADMINISTRATION.

#A. TRAFFIC CHARACTERISTICS :-

- characteristics of traffic are further governed by
 - (i) Road user characteristics.
 - (ii) vehicular characteristics.

(i) ROAD USER CHARACTERISTICS :-

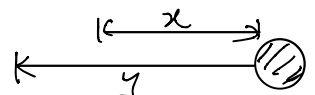
- It involves all actions of road user either as pedestrian, cyclist, car driver or motorist or PCU driver.
- The various factors which affects the road user actions are as follows :-

(a) PHYSICAL :- physical characteristics of road user may either be permanent or temporary

- Permanent characteristics :- VISION, HEARING, STRENGTH, GENERAL REACTION to situation.

Eg, $\text{vision} = \frac{x}{y} = \frac{\text{Actual distance}}{\text{Required distance}} \rightarrow$ For a normal eye

6/6 vision or Normal vision



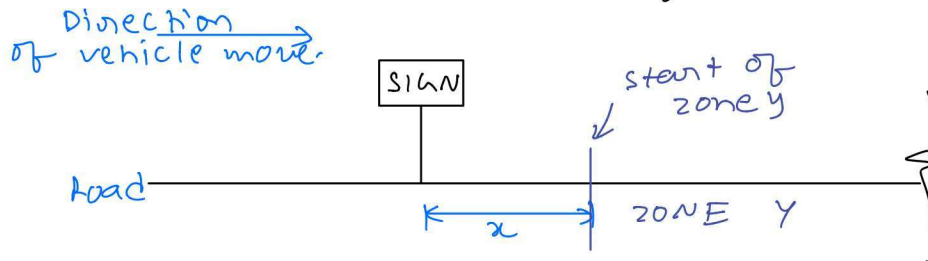
It is an ability of a person to recognize a letter of size 8.5mm from a distance of 6m

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6/9 vision :- This vision corresponds to poor vision. here person is able to recognize the object (8.5mm letter) from dist. of 6m instead of 9m.

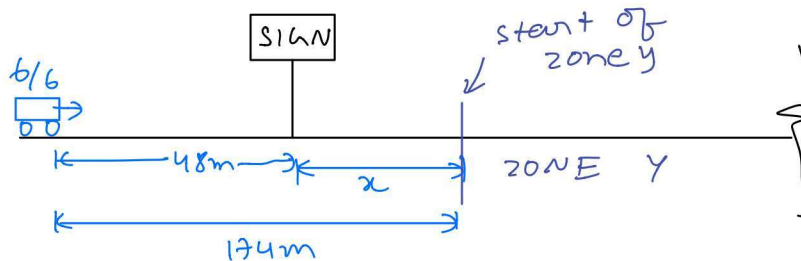
10/6 vision :- Super vision. (Practically it is not possible).

- Q) A sign is required to be put up asking driver to slow down to 30 kmph before entering zone Y. On this road vehicles require 174 m to slow down to 30 kmph (the distance of 174m includes the distance travelled during the perception-reaction time of the driver). The sign can be read by 6/6 vision drivers from a distance of 48m. The sign is placed at a distance "x" from the start of zone Y so that even a 6/9 vision driver can slow down to 30 kmph. before entering the zone. The min. value of x = ?.



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Sol



$\frac{6}{6}$ vision - 48 m

9m - 6m

1m - $\frac{6}{9}$ m

48m - $\frac{6}{9} \times 48$
= 32m

$x = 174 - 32$
= 142m.

- Temporary characteristics :- Affecting their efficiency are **FATIGUE, DRUGS INTAKE,**

ILLNESS

- All these will increase the reaction time of driver.

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(b) MENTAL FACTORS :- It includes, knowledge, skill, intelligence, experience and literacy of road user which governs its behaviour on road

(c) PSYCHOLOGICAL FACTORS :- These effects reaction to traffic situations of road users to great extent.

- It includes attentiveness, fear, anger, superstition, ill patients, general attitude towards traffic & regulations.

(d) ENVIRONMENTAL FACTORS :- It includes, traffic stream characteristics, road side feature, atmospheric condition and locality.

(ii) VEHICULAR CHARACTERISTICS :-

- These characteristics are further classified into
(a) static characteristics.
(b) dynamic characteristics

(a) STATIC :- static charact. which affect the road design are :-

- 1) Dimension $\rightarrow$ length, Breadth, height.
- 2) Turning angle.
- 3) Total wt. of vehicle.
- 4) Ground clearance.

(i) DIMENSION :- length of the vehicle will control horizontal alignment of the road, extra widening, parking facilities,

- height of vehicle controls height of bridge, height of cables, lines, other service line, overhead sign boards, under pass.
- width of vehicle governs lane width, shoulder width, parking design.

(ii) Turning angle :- governs curve resistance, Radius,

(iii) wt. of vehicle :- governs thickness of pavement, gradient of road.

(iv) Ground clearance :- decides the ht. of bumps, approaches i.e. ramps.

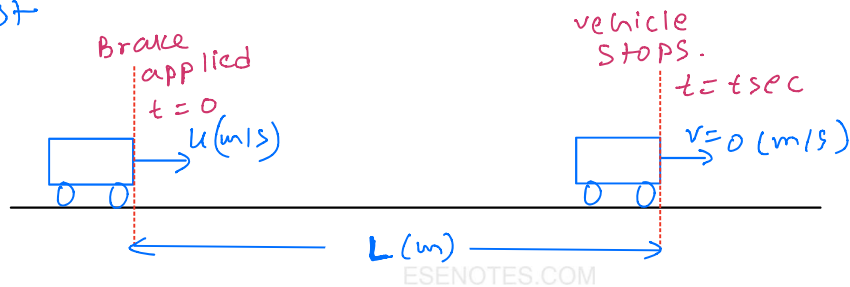
(b) DYNAMIC :- characteristics includes :-

- Power of vehicle decides gradient of road, hauling capacity, rolling resistance.
- Speed of vehicle governs geometric design elements

(c) BRAKING :- governs sight distance, efficient braking, deceleration, impact on road material.

Jan 19

- Braking characteristics of vehicle depends on design and type of braking system
Eg - Mechanical, fluid, air brakes.
which can be determined by performing brake test



In this test brakes are applied till the vehicle completely stops and two of the three parameters are noted to find skid resistance.

- (a) Braking distance (L)
- (b) Initial speed (u)
- (c) Actual duration of Brake application (t).

(i) Retardation : $v = u + at$
 $0 = u + at$
 $a = \ominus (u/t) \rightarrow \text{deceleration.}$

(ii) Braking distance :- $L = ut + \frac{1}{2} at^2$
 $L = ut + \frac{1}{2} \left(\frac{-u}{t} \right) t^2 = ut - \frac{ut}{2} = \frac{ut}{2}$
 $L = \left(\frac{ut}{2} \right) = \frac{1}{2} \times \text{initial speed} \times \text{duration of brake.}$

(iii) Skid Resistance :- $v^2 = u^2 + 2aL$
 $0 = u^2 + 2 \left(\frac{-u}{t} \right) L$
 $L = \frac{u^2 t}{2u} = \frac{ut}{2} \quad \text{--- (i)}$

$\Delta KE = \text{work done}$

$$\begin{aligned}\frac{1}{2}mv^2 - \frac{1}{2}mu^2 &= \ominus F \cdot L \quad \text{force and displacement are opposite.} \\ -\frac{1}{2}mu^2 &= -F \cdot L \\ \frac{1}{2}mu^2 &= -fmgL \quad f = fmg \\ L &= \frac{u^2}{2gf} \quad \text{--- (ii)}\end{aligned}$$

from (i) & (ii)

$$\frac{ut}{2} = \frac{u^2}{2gf}$$

$$f = \frac{u}{gt} = \frac{-a}{g} = \frac{\text{retardation}}{\text{acc. due to gravity}}$$

- Braking efficiency, $\eta_b = \frac{\text{Apparent friction}}{\text{Max friction}} = \frac{f}{f_{\text{max}}} \times 100$
apparent means actual friction.

- Q) A vehicle moving at 50 km/h speed was stopped by applying brakes and length of skid mark was 16.5 m. If the avg skid resistance of the pavement is known as 0.687. what is the brake eff. of the test vehicle.

Sol

$$\begin{aligned}v &= 50 \text{ kmph} \\ L &= 16.5 \text{ m} \\ f &= 0.687\end{aligned}$$

skid res's tance developed

$$f = \frac{-a}{g} = \frac{u}{gt} = \frac{u \times u}{g \cdot 2L} = \frac{u^2}{2gL}$$

$$f = \frac{(0.278 \times 50)^2}{2 \times 9.81 \times 16.5} = 0.59$$

$$\eta_B = \frac{0.59}{0.687} \times 100 = 86.7\%$$

NOTE:- if $\eta_B = 100\%$, it means wheel is locked

- Q) Find out coeff of friction if,

(a) Braking distance is 5.8 m & speed of vehicle is 30 kmph.

(b) Initial speed of vehicle is 40 kmph and duration of brake application is 1.8 sec.

(c) Braking distance is 7m & duration of brake application is 1.4 sec

Solvik 2025 Sol (a) $L = 5.8 \text{ m}$, $v = 30 \text{ kmph}$

$$L = \frac{u^2}{2gf} \quad \therefore f = \frac{(0.278 \times 30)^2}{2 \times 9.81 \times 5.8} = 0.611$$

(b) $v = 40 \text{ kmph}$
 $t = 1.8 \text{ s}$

$$f = \frac{u}{gt} = \frac{0.278 \times 40}{9.81 \times 1.8} = 0.629$$

(c) $L = 7 \text{ m}$
 $t = 1.4 \text{ s}$

$$f = \frac{u}{gt} = \frac{2L}{gt^2} = \frac{2 \times 7}{9.81 \times 1.4^2} = 0.728$$

B TRAFFIC STUDIES & ANALYSIS

- TE studies are carried out for collecting traffic data, for its analysis to identify the movement of traffic along the particular road so as to apply it for traffic control measures and efficient UD.
- The different types of traffic engineering studies are as follows:-

- TRAFFIC VOLUME STUDIES.
- TRAFFIC & CAPACITY ANALYSIS.
- SPOT SPEED STUDIES.
- ORIGIN & DESTINATION (O&D) STUDIES.
- PARKING STUDIES.
- ACCIDENT STUDIES.

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TRAFFIC VOLUME STUDY :-

- Traffic volume is a measure to quantify the traffic flow and is expressed as no. of vehicles that pass across a given section of road during unit time.
- As the carriageway width of the road may vary the traffic volume is expressed as number of vehicles per unit time per traffic lane.
- Different classes of vehicle make use of same road hence for mixed flow traffic, different classes of the vehicle must be transformed, expressed or converted into one standard type of unit such as "passenger car."
- Hence each vehicle class is assigned an equivalent factor termed as passenger car unit (PCU) which is defined as

$$[PCU]_i = \frac{\text{Speed Ratio}}{\text{Length Ratio}} = \frac{(V_{su}/V_i)}{(L_{su}/L_i)}$$

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PCU values recommended by IRC for diff. vehicles on urban roads!!

VEHICLE TYPE	EQUIVALENT PCU
1) Two wheeler: Scooter, Bike	0.7
2) Passenger car: van	1
3) Auto Rickshaw	2
4) LCV	2
5) Truck / Bus.	3.7
6) Tractor trailer	5

- Traffic volume studies are being carried out:
 - to decide the priority
 - for improvement of design.
 - for computing roadway capacity
 - to plan traffic operations and control.

• Traffic volume can be measured by any of following mtd.

(i) MANUAL METHOD:- very accurate method in which different class of vehicle can also be noted, but cannot be used for continuous measurement.

(ii) MECHANICAL MTD:-

- RADAR. (Radio detection & ranging)
- PNEUMATIC SENSORS. (Based on pressure applied)
- MAGNETIC DETECTOR.
- CCTV cameras.

- Traffic volume data can be represented in any of following forms.

(i) AADT (Annual avg daily traffic)

$$AADT = \frac{\text{Total no. of vehicles throughout a year}}{365} \quad \text{veh/day}$$

(ii) AAMT (Annual avg hourly traffic).

$$AAMT = \left[\frac{\text{Total no. of vehicle through year}}{365 \times 24} \right] \quad \text{veh/hr.}$$

$$AAMT = \frac{AADT}{24}$$

(iii) AAWT (Annual avg weekday traffic)

$$AAWT = \frac{\text{Total no. of vehicle throughout week}}{52 \times 5 = 260} \text{ ven/day.}$$

AAWT > AADT weekday = Monday to Friday.
on weekend volume is considerable reduced

(iv) ADT (Average daily traffic).

$$ADT = \left[\frac{\text{Total number of vehicles throughout a week}}{7} \right]$$

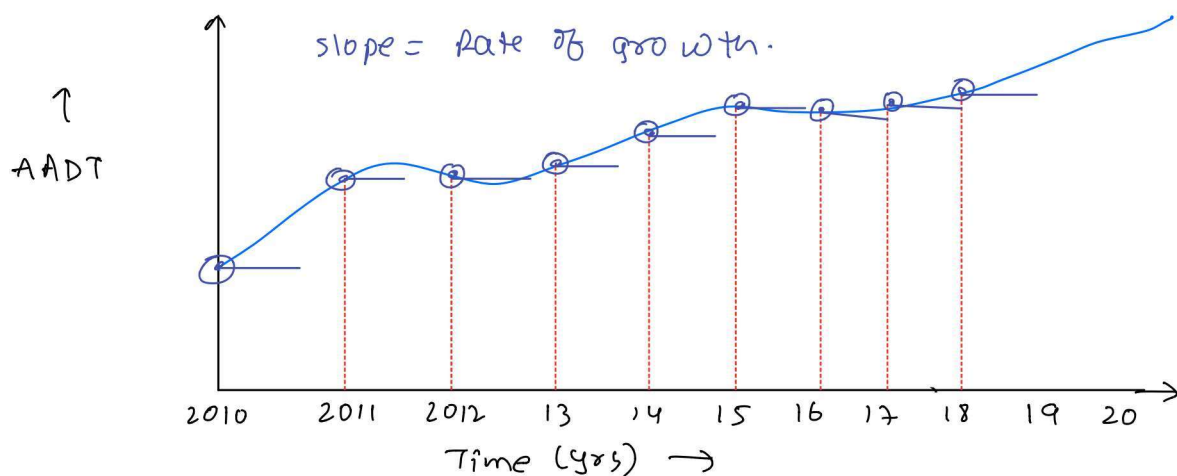
ADT > AADT → Jitra bada sample size utna chota fluctuation

Jan 22 (v) TREND CHART :- Traffic volume data can be represented in form of trend chart which is useful in estimating the rate of growth of traffic of a particular class that can be used for planning future expansion design and regulation.

- If from trend chart rate of growth of traffic volume is computed to be $n\%$, then future traffic is given by.

$$Q_n = Q_0 (1+n)^n$$

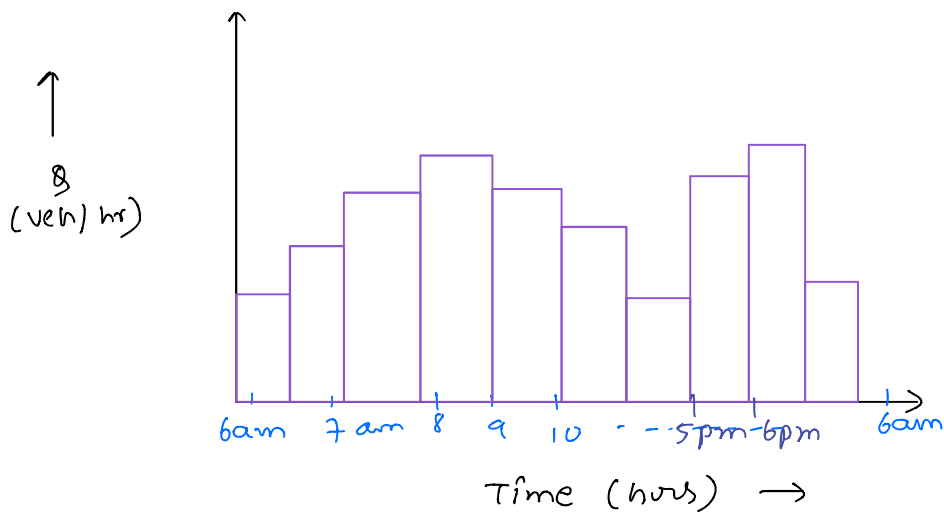
Q_0 = present traffic volume.
 n = rate of growth
→ avg of all rate of growth



(vi) VARIATION CHART :-

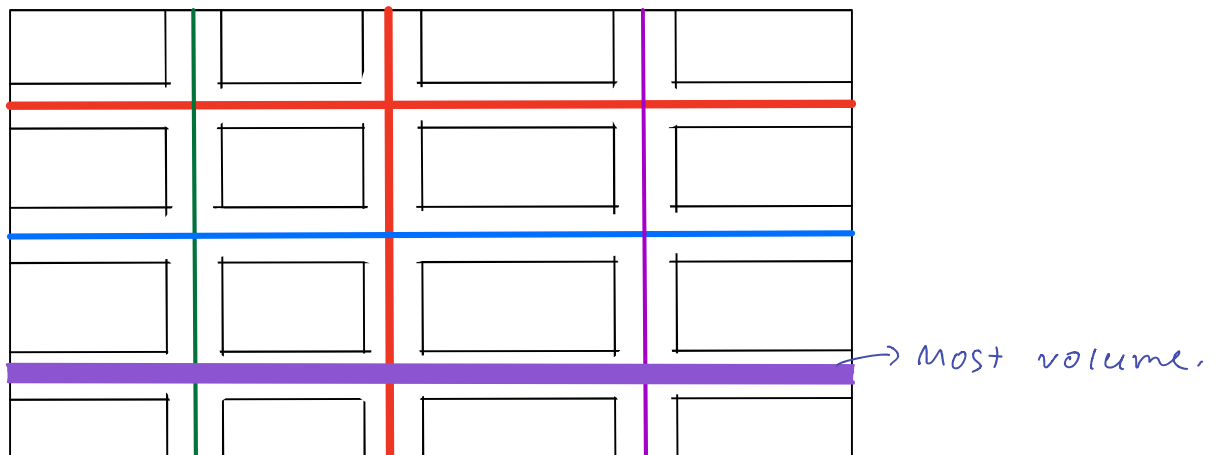
Souvik 2021 - variation chart can be used to show, hourly, daily or seasonal variation of traffic volume, over a particular section of road.

- This chart helps in deciding the facilities and regulation needed during peak traffic period.



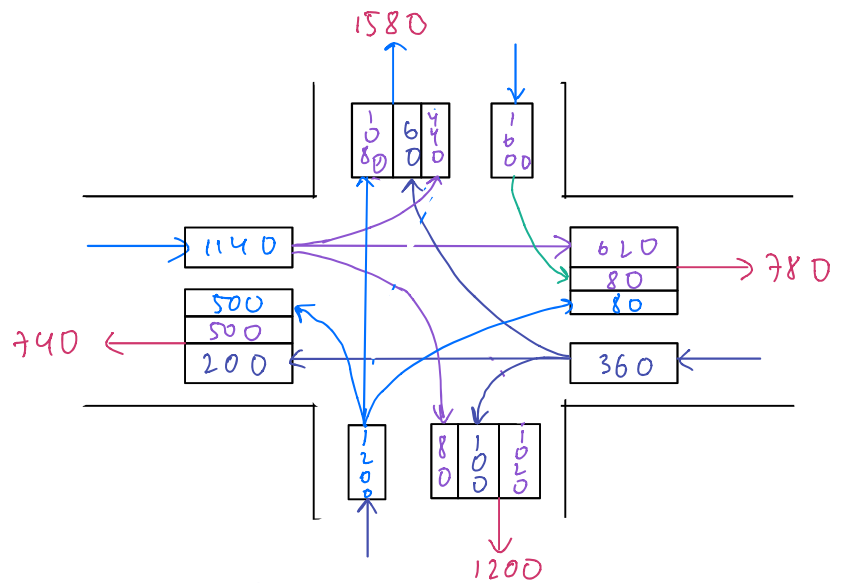
(vii) TRAFFIC FLOW MAPS :-

- These maps along the routes indicates the vol. of traffic flowing thru them.
- The thickness of line represents the traffic volume to any desired scale.
- This helps to find the traffic volume distribution along the existing roads at a glance.

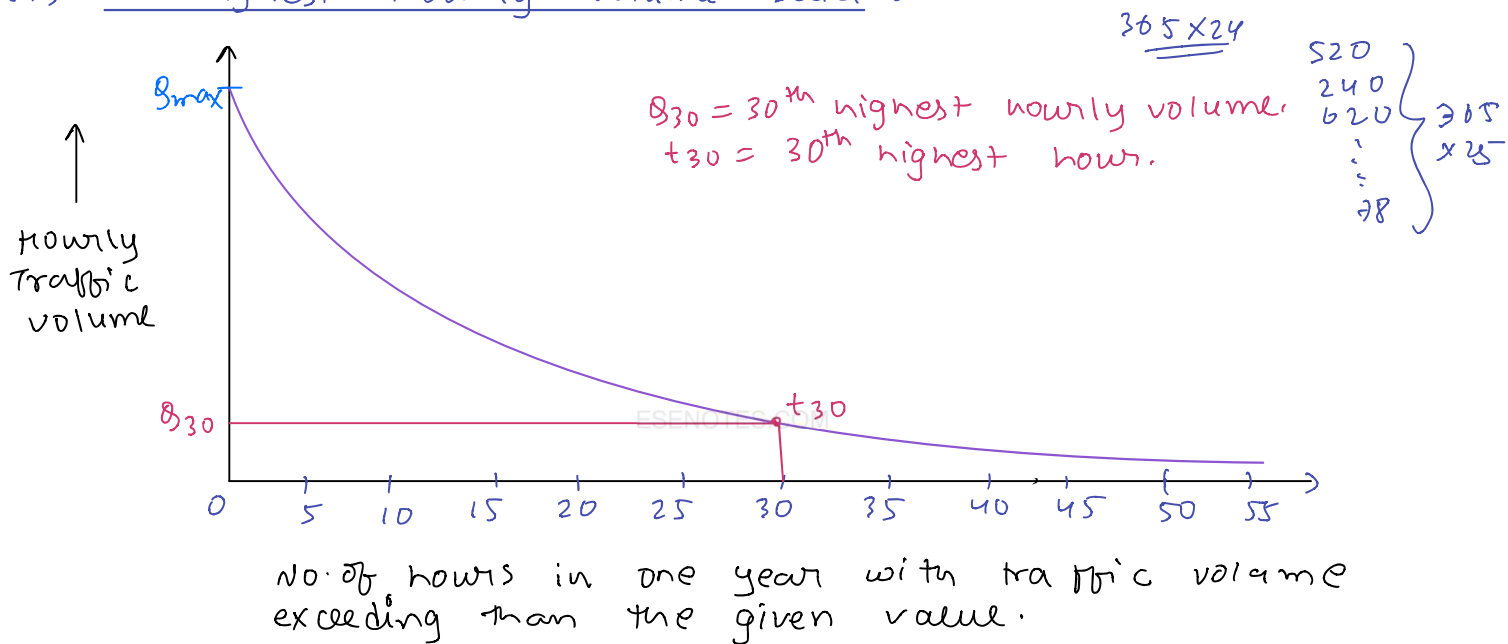


(viii) volume flow diagram at intersection :-

- volume flow diagram at intersection either drawn to be certain scale or indicating traffic volume can be used to find the details of crossing and turning traffic.
- These data are required for design of Intersection.



(ix) 30th Highest Hourly volume Data :-



- If designing of a road is done for highest or peak hourly volume of a year, it would be highly uneconomical (S_{max}).
- However if it is done for (AAHV) it would not be sufficient for considerable period of year to carry the traffic volume and would lead to congestion of the road.
- 30th highest hourly traffic volume is the hourly volume that will be reached only 30 times or exceeded only 29 times in a year.
- As per IRC if designing is done for 30th HHV, then neither it is costly, nor it is congested.
- Hence eliminates the problem of congestion for most part of the year. % time congestion = $\frac{29}{365} \times 100 = 0.33\%$

NOTE:- PERIODIC VOLUME COUNT

- In order to find certain traffic data such as AADT, ADT etc it is necessary to count traffic for long duration of time may be for years and time.
- counting traffic for long duration of time involves cost and effort which is not feasible practically. In such case IRC suggest, to make use of technique of periodic volume count.
- In volume count, expansion factors are used as follows :-

(a) Hourly expansion factor :-

$$HEF = \left[\frac{\text{Total traffic for a day}}{\text{Traffic for a particular hour.}} \right]$$

(b) Daily expansion factor :-

$$DEF = \left[\frac{\text{Traffic for a week}}{\text{Traffic for a particular day}} \right]$$

$$\text{Traffic for a week} = DEF \times HEF \times \text{Traffic for a particular hour}$$

(c) Monthly expansion factor :-

$$MEF = \left[\frac{AADT}{ADT} \right]$$

- Q) A traffic engg. needs to find AADT on a road based on a data on wednesday during month of june. Determine AADT, if daily expansion factor is 7.9 and monthly expansion factor for june is 1.5.

Time Period	Vehicles	HEF
8-9 am	500	25
9-10 am	600	22
10-11 am	700	20

Sol For 8-9 am

$$HEF = \frac{\text{one day traffic}}{\text{particular hr vol.}}$$

$$\text{one day traff} = 25 \times 500 = 12500$$

$$\text{1 week traff} = 7 \cdot 9 \times 12500 = 98750$$

$$ADT = \frac{98750}{7} = 14107.14$$

$$AADT = 1.5 \times 14107.14 = 21160.71$$

For 9-10 am

$$AADT = 1.5 \times \frac{7 \cdot 9 \times 22 \times 600}{7} = 22345.71.$$

For 10-11 am

$$AADT = 1.5 \times \frac{7 \cdot 9 \times 20 \times 700}{7} = 23700$$

$$AADT = 22402$$

(d) Peak hourly factor (PHF)

- PHF may be expressed in terms of traffic volume during peak hour and is used in design of transportation facilities and infrastructure projects

$$V_n(\text{max}) = \frac{\text{Peak hourly flow} \times n}{60 \times (PHF)_n}$$

$$(PHF)_n = \frac{\text{Peak hour flow}}{\left(\frac{60}{n}\right) \times V_{n,\text{max}}} \quad \frac{n}{60} \leq (PHF)_n \leq 1$$

- 8) The minute flow value were observed for the peak hour on an expressway section as shown. Determine 5 min & 15 min peak hour factor.

Minute (i)	1-7	8-10	11-20	21	22-32	33-45	46-47	48-55	56-60
N(i)	20	30	25	26	4	10	15	30	10

$$\text{peak hourly flow} = 7 \times 20 + 1 \times 30 + 10 \times 25 + 1 \times 26 + 11 \times 4 + 13 \times 10 + 2 \times 15 + 8 \times 30 + 5 \times 10$$

$$= 1000 \text{ veh/hr.}$$

Sol
Souvik
2021

$$(i) \quad PHF_5 = \frac{\text{Peak hourly flow}}{\frac{60}{5} \times v_{5, \max}} = \frac{1000}{12 \times 150} = 0.55$$

$$v_{5, \max} = 5 \times 30 \rightarrow (48-55) \text{ min} \\ = 150$$

$$(ii) \quad PHF_{15} = \frac{\text{Peak hourly flow}}{\frac{60}{15} \times v_{15, \max}}$$

$$v_{15, \max} = 7^{\text{th}} \text{ min to } 21^{\text{th}} \text{ min} \\ = 1 \times 20 + 3 \times 30 + 10 \times 25 + 1 \times 26 \\ = 386 \text{ veh}$$

$$PHF_{15} = \frac{1000}{\frac{60}{15} \times 386} = 0.647$$

Jan 23

(III) TRAFFIC FLOW & CAPACITY ANALYSIS :-

- Significant terms used in traffic flow and capacity analyses.

(a) Traffic Density (K) :- Number of vehicles occupied by unit length of the road is called traffic density.

- It is expressed as **vehicle/km**

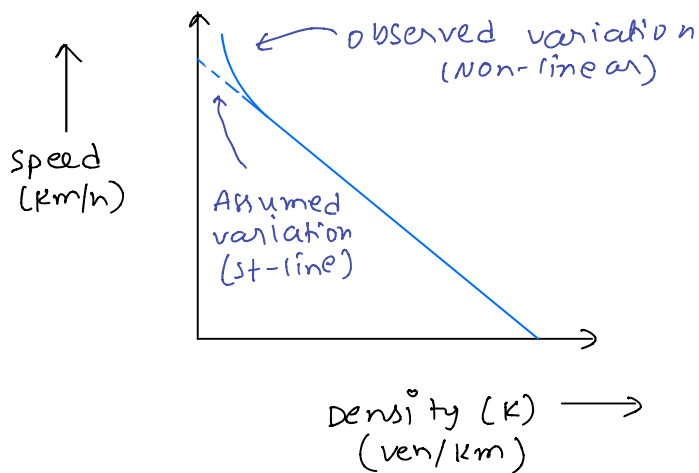
- The max. value of "K" which a road can accommodate is termed as "Jam density" or Traffic density under jam condition is termed as Jam density.

- Under Jam condition, the velocity will be zero.

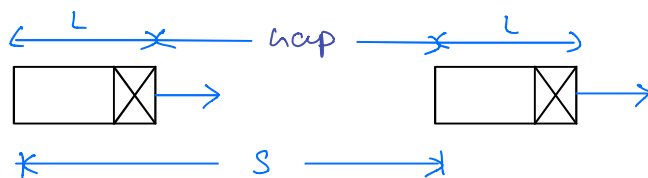
- It is the worst condition of the road.

- Traffic density varies with speed inversely, hence with increase in speed of a stream vehicles on a roadway, the average density of vehicle decreases.

- This is because gap or spacing b/w the vehicles increases



(b) Space Headway (S/m) It is the distance maintained b/w 2 consecutive vehicle travelling in same directions.



- It is the total length occupied by a moving vehicle on road which includes length of vehicle and the gap b/w the two vehicles.
- This gap for safety point of view must be equal to SSD to avoid the collision b/w the two vehicles, but this value of gap makes the designing over safe (capacity reduces).
- Hence for geometric design this gap is computed by neglected braking distance & considering reaction time to be 0.7 sec instead.

makes own ← designing over safe

$$S = SSD + l$$

$$S = 0.278 V_D t_r + \frac{V^2}{254 f} + l$$

$t_r = 2.5 \text{ sec.}$

(**) If "f" is given in question use SSD only.

For geometric design

we make Braking distance = 0 and $t_r = 0.7 \text{ s}$
 $l = 6 \text{ m.}$

$$S = 0.278 \times 0.7 V_D + 6$$

$$S = 0.2 V + 6 \quad V \rightarrow \text{Kmph} \quad (S \rightarrow \text{m})$$

Hence Traffic density (K) = $\frac{\text{length of road (1km)}}{\text{Space head way (S)}}$

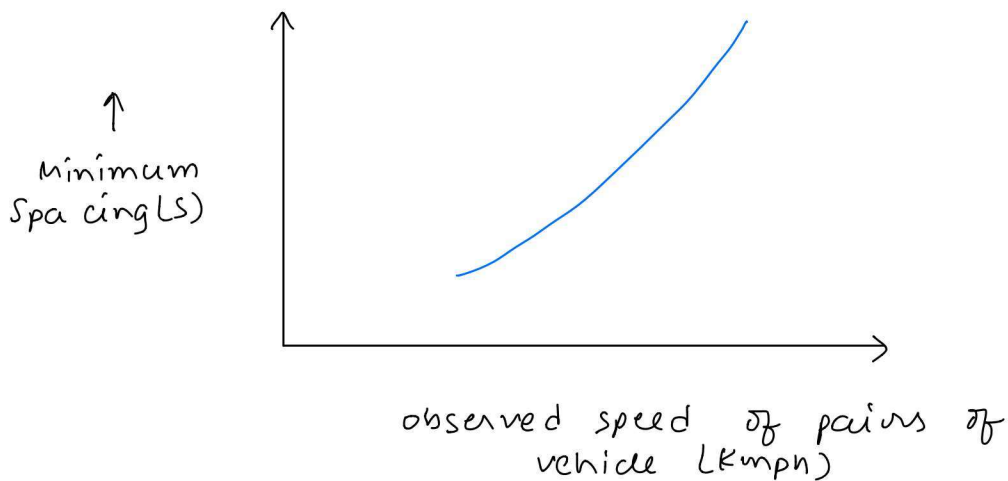
$$k = \frac{1000}{S(m)} \quad (\text{veh/km})$$

$$k = \frac{1000}{0.2v + l}$$

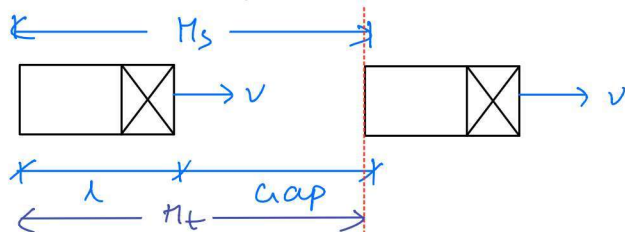
NOTE :- Under Jam condition $v = 0 \text{ kmph}$

Jam density, $k_j = \frac{1000}{l(m)} \quad l = \text{length of wheel base} = 6m$

$$= 166 \text{ veh/km}$$

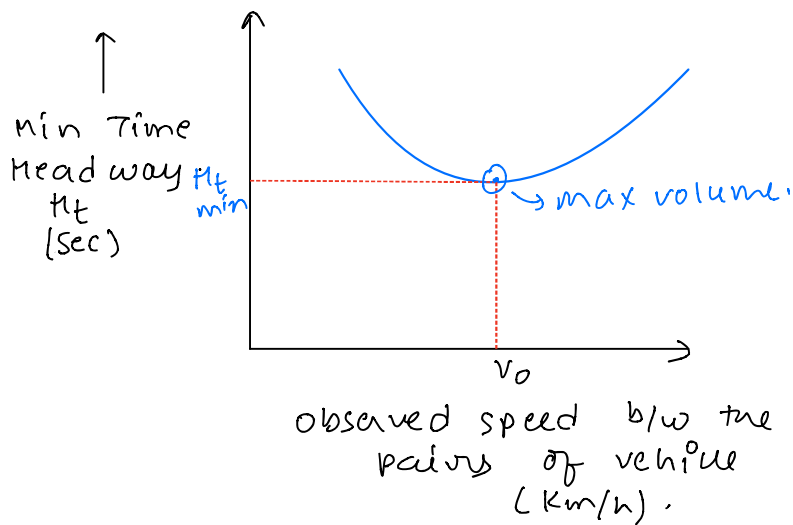


(c) Time Headway (H_t) :-



- It is the time gap b/w passes of two consecutive vehicles.
- Assume that a stream of vehicle flows at close interval on behind the other.
- At very low speed, time headway is high and no. of vehicles crossing a section on the road is also low.
- As the speed of stream increases, the time headway decreases up to a lowest value at a certain speed.
- The speed at which the value of time headway is lowest represents the optimum speed corresponding to maximum flow or termed as "CAPACITY FLOW"

- If speed of traffic stream is further increased the min time headway starts increasing resulting in decrease of traffic flow.



Hence Traffic volume $q = \frac{1 \text{ hr}}{H_t} = \frac{3600}{H_t (\text{Sec})} \text{ veh/hr}$

$$q = \frac{3600 (\text{Sec})}{H_t (\text{Sec})}$$

Relation b/w Traffic volume (q), Density (K) & speed (V).

- It is difficult to measure traffic density directly in practical situation, hence the relationship b/w volume density and speed is used.

$$K (\text{veh/km}) = \frac{q (\text{veh/hr})}{V (\text{km/hr})}$$

$$\therefore q = KV$$

Traffic volume = Traffic density $\times$ speed.

$$q = \frac{1000 \cdot V}{H_s} \quad \begin{matrix} S \rightarrow (\text{m}) \\ V \rightarrow (\text{kmph}) \end{matrix}$$

(veh/hr)

Also, $q = \frac{3600}{H_t}$

from above two equation

$$\frac{1000 V}{H_s} = \frac{3600}{H_t}$$

$$\frac{H_s}{H_t} = \frac{V}{3.6}$$

(m) $\leftarrow$ $\frac{H_s}{H_t}$ $\rightarrow$ (kmph)
(s)

Note:-1) Max value of traffic volume, which a road can accommodate is called **Traffic capacity**.

2) This traffic capacity under most ideal condition is termed as **BASIC capacity / Theoretical capacity**.

8) A traffic stream in a particular direction of a two lane road is moving with a const. speed of 50 kmph, with an average headway of 2.52 sec. The longitudinal distance b/w two consecutive vehicle would be.

Sol
 $V = 50 \text{ kmph}$
 $H_t = 2.52 \text{ sec}$

$$\text{Density } K = \frac{1000}{H_s}$$

$$\text{volume} = \frac{3600}{2.52} = 1428.57 \text{ (veh/hr)}$$

$$q = KV$$
$$1428.57 = \frac{1000}{H_s} \times 50$$
$$H_s = 35 \text{ m}$$

8) In a one lane one way homogenous traffic stream the observed avg headway is 3 sec. The flow in this traffic stream would be.

Sol :- $H_t = 3 \text{ sec}$

$$\text{Flow, } q = \frac{3600}{3} = 1200 \text{ (veh/hr / lane)}$$

8) A single lane uni-directional highway was designed with speed of 60 kmph. The perception brake Reaction time = 2.4 sec and the avg length of wheel base = 5m. The coeff of long. friction is 0.38. The capacity of this road in terms of vehicle per hr per lane would be.

Sol
 $V_d = 60 \text{ kmph}$ $t_{br} = 2.4 \text{ sec}$ $L = 5 \text{ m}$

$$\text{Space headway, } S = 0.278 V_d t_{br} + \frac{V^2}{254 f} + L$$

$$S = 0.278 \times 60 \times 2.4 + \frac{60^2}{254 \times 0.38} + 5$$

$$S = 82.32 \text{ m}$$

$$\text{Capacity} = \frac{1000}{H_s} V = \frac{1000 \times 60}{82.33} = 728.8 \text{ (V/hr/lane)}$$

- 8) The avg spacing b/w vehicles in a traffic stream is 50 m, then the density of the stream would be?

Sol $H_s = 50 \text{ m}$

Density, $k = \frac{1000}{50} = 20 \text{ veh/km}$.

Jan 24 (IV) TRAFFIC FLOW MODELS :-

- To figure out the exact relationship b/w the traffic parameters a number of research is being carried out.
- The result of these researches yielded many mathematical models broadly classified into following

(a) MACROSCOPIC MODEL

(b) MICROSCOPIC MODEL.

(a) MACROSCOPIC MODEL :-

- It represents how the behaviour of one parameter of traffic flow changes w.r.t another.
- Following types of macroscopic models are available.

(i) GREENSHIELD'S MODEL :-

- Green shield assumed a linear speed density relationship to derive this model.

$v_f \rightarrow$ free speed @ $k=0$.

$k_j \rightarrow$ Jam density @ $v=0$.

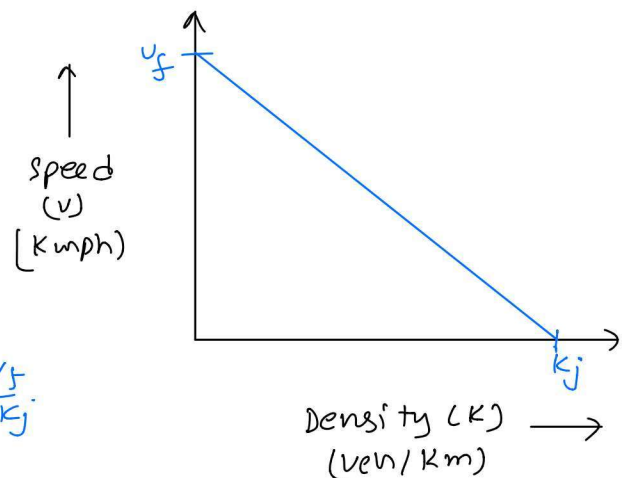
eg. $y = mx + c$

$v = m k + v_f$

$m = \frac{y_2 - y_1}{x_2 - x_1} = \frac{0 - v_f}{k_j - 0} = - \frac{v_f}{k_j}$

$v = - \frac{v_f}{k_j} k + v_f$

$v = v_f \left(1 - \frac{k}{k_j}\right)$

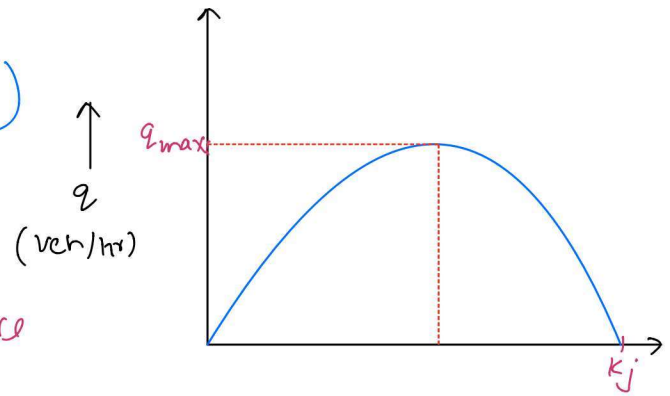


$\rightarrow$ This is known as "Green shields model". very old theory.

Now, $q = kv$
 $q = k v_f \left(1 - \frac{k}{k_j}\right)$

$$q = v_f k - \frac{v_f k^2}{k_j}$$

→ Think of the curve with speed, time headway & space headway.



Using the above relation, density can be found at which max flow occurs. (veh/km)

$$\frac{dq}{dk} = v_f - \frac{2v_f k}{k_j} = 0$$

$$\frac{2k}{k_j} = 1$$

$$k = \frac{k_j}{2}$$

- Density at which max flow occurs is denoted as $k_0 = k_j/2$ k_0 = optimum density.

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$$q_{max} = v_f k_0 - \frac{v_f k_0^2}{k_j}$$

$$q_{max} = v_f \frac{k_j}{2} - \frac{v_f k_j^2}{4k_j} = v_f \frac{k_j}{2} - \frac{v_f k_j}{4}$$

$$q_{max} = \frac{v_f k_j}{4}$$

→ As per Green shield model.

- Since $q = v_f k - \frac{v_f k^2}{k_j}$

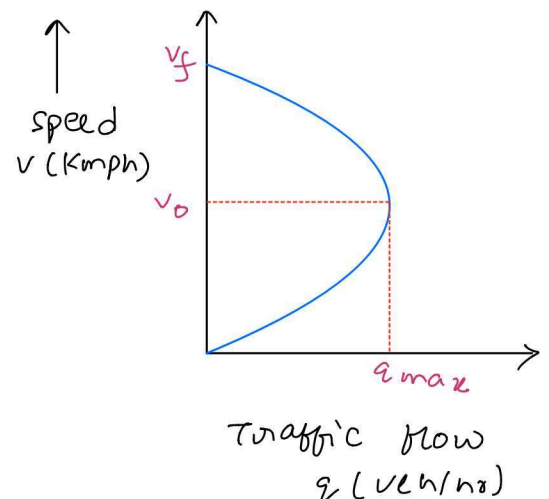
$$q = kv \Rightarrow k = q/v$$

$$q = v_f \frac{q}{v} - \frac{v_f q^2}{v^2 k_j}$$

$$1 = \frac{v_f}{v} - \frac{v_f q}{v^2 k_j}$$

$$k_j v^2 = v k_f k_j - v_f q$$

$$q = k_j v - \left(\frac{k_j}{v_f}\right) v^2$$



To find the speed at which flow is max.

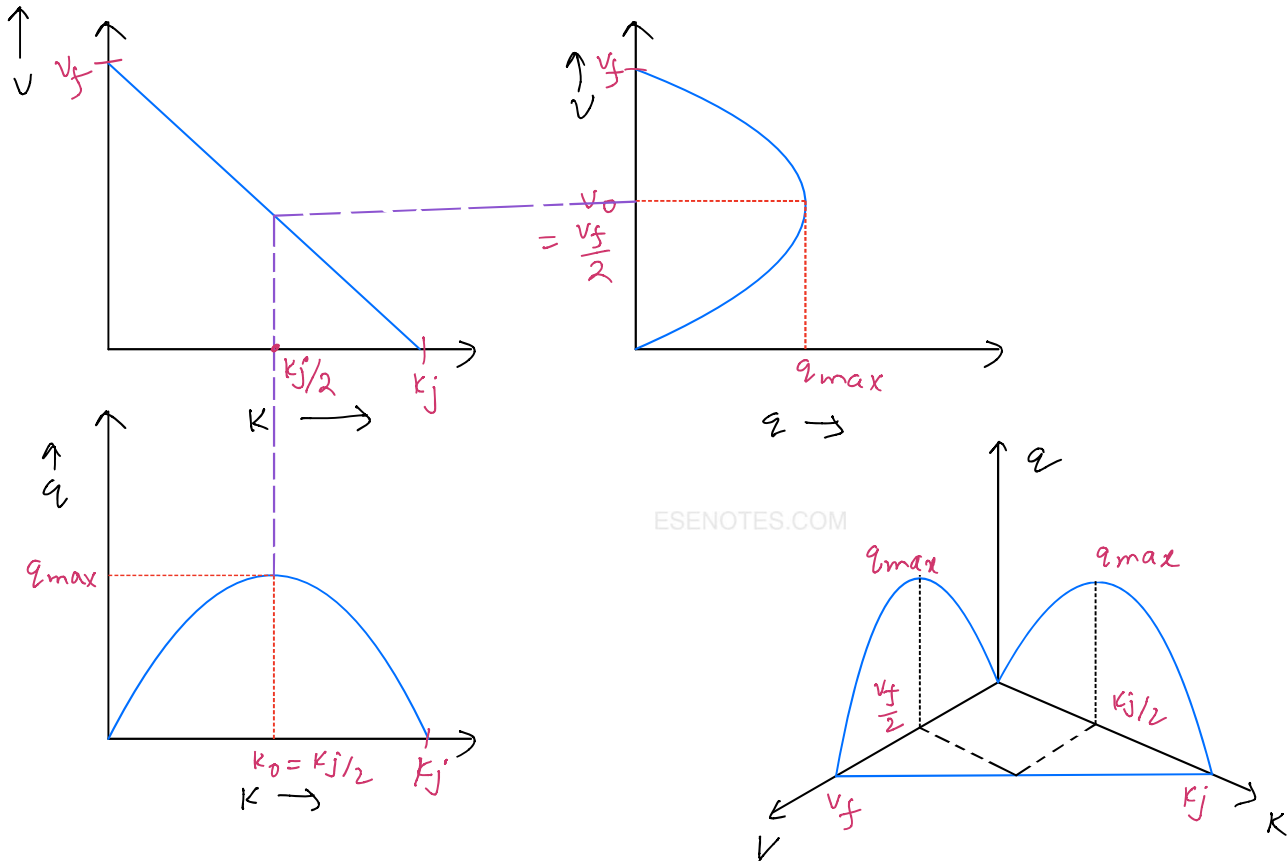
$$K = K_0 = \frac{K_j}{2}$$

$$v = v_f \left(1 - \frac{K}{K_j}\right)$$

At optimum speed $v = v_0$, $K = K_j/2$

$$v_0 = v_f \left(1 - \frac{1}{2}\right)$$

$$v_0 = \frac{v_f}{2}$$



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NOTE:- In this model linear relationship b/w speed & density is assumed, which is rarely found in actual, hence several other models are developed to overcome this drawback.

Q) For a traffic stream, speed, density relationship was found to be

$$u = 42.46 - 0.22K$$

$u \rightarrow$ speed in kmph

$K \rightarrow$ density in (veh/km)

Determine

(i) Free mean speed

(iv) speed at max flow

(ii) Density at max flow. (v) Max flow.

(iii) Jam density

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sol/ (i) Free mean speed, $v_f \big|_{k=0} = 42.46 \text{ kmph.}$

(ii) Jam density, $k_j \big|_{v=0} = \frac{42.46}{0.22} = 193 \text{ veh/km.}$

(iii) Density at max flow. $k = \frac{k_j}{2} = \frac{193}{2} = 96.5 \text{ veh/km}$

(iv) Speed at max flow, $v_0 = \frac{v_f}{2} = \frac{42.46}{2} = 21.23 \text{ kmph}$

(v) Max flow, $q = kv$

$$q_{\max} = \frac{k_j}{2} \cdot \frac{v_f}{2} \\ = 96.5 \times 21.23 = 2048.7 \text{ veh/hr.}$$

(vi) Space headway at max flow.

$$k = \frac{1000}{h_s}$$

$$h_s = \frac{1000 \times 2}{k_j} = \frac{2000}{193} = 10.36 \text{ m}$$

(vii) Time headway at max flow.

$$q = \frac{3600}{h_T}$$

$$h_T = \frac{3600}{2048.7} = 1.75 \text{ sec}$$

(viii) Avg length occupied by vehicle in Jam condition

$$(h_s)_{\text{jam}} = \frac{1000}{k_j} = \frac{1000}{193} = 5.18 \text{ m}$$

(ii) GREENBERG'S LOG-MODEL :-

- Greenberg used a fluid flow analogy concept & proposed a logarithmic speed density relationship
 - The greatest advantage of this method is that it can be derived analytically.
 - This method shows better fit as compared to green shield model but the drawback of this model is its inability to predict the speed at lower density as speed approaches infinity at lower density. $\rightarrow$ drawback.
- $\rightarrow$ It cannot be used where k is low and v is high.

As per this model,

$$v = v_0 \ln\left(\frac{k_j}{k}\right)$$

v = speed
 v_0 = optimum speed
 k_j = Jam density

Now, $q = kv$

$$q = k v_0 \ln\left(\frac{k_j}{k}\right) = k v_0 (\ln k_j - \ln k)$$

$$\frac{dq}{dk} = 0 \text{ maximize}$$

$$\frac{dq}{dk} = v_0 (\ln k_j - \ln k) + k v_0 \left(-\frac{1}{k}\right)$$

$$(k_0) \leftarrow k_0 = \frac{k_j}{e} \quad k_0 \rightarrow \text{optimum density.}$$

$$q_{\max} = \frac{k_j}{e} v_0 (\ln k_j - \ln \frac{k_j}{e})$$

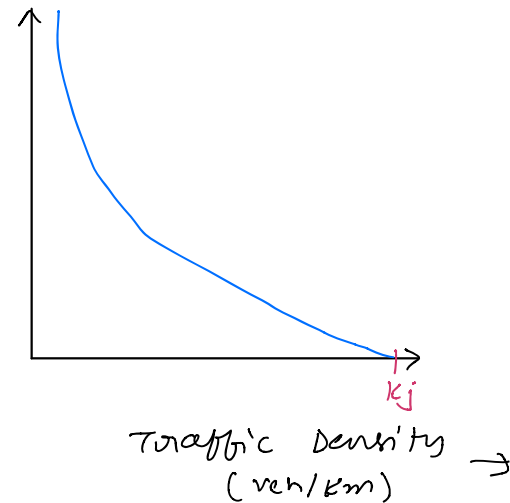
$$= \frac{k_j}{e} v_0 (\ln k_j - \ln k_j + 1)$$

$$q_{\max} = \frac{v_0 k_j}{e}$$

At $q_{\max}$, $v = v_0$, $k = k_0$

$$v = v_0 \ln \frac{k_j}{k}$$

$$v = v_0$$



Jan 25 Q) On a specific highway, the speed density relationship follows greenberg's model. The free flow velocity is 80 kmph. and jam density is 200 veh/km. when the highway is operating at max capacity, the density obtained as per this model would be? (veh/km) and the value of max flow.

Sol $v_f = 80 \text{ kmph.}$ $k_j = 200 \text{ veh/hr.}$

$$v_0 = \frac{v_f}{2} = 40 \text{ kmph} \quad \text{At max cap. } k = k_0 = \frac{k_j}{e}$$

$$k_0 = 73.8 \text{ veh/hr}$$

$$q_{\max} = \frac{v_0 k_j}{e} = \frac{40 \times 200}{e} = 2943 \text{ veh/hr.}$$

(iii) UNDERWOOD EXPONENTIAL MODEL :-

- It is the modification of greenberg's model which was unable to represent velocity of stream at lower density.
- Underwood hence proposed exponential model for traffic flow as follows :-

$$v = v_f e^{-\frac{k}{k_0}}$$

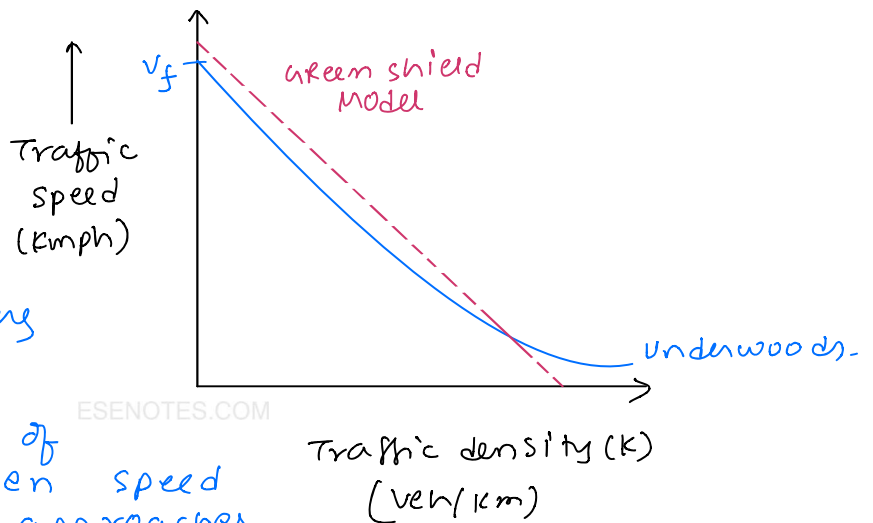
$$k_0 = k_j/2 \quad \leftarrow$$

v_f = free mean speed (kmph)

k = Density

k_0 = optimum density i.e density corresponding to max flow

- underwood model tends to underestimate speed at free flow phase but overestimate the congested position. (This is just opposite of analysis in greenberg model)



- The main drawback of this model is when speed become zero, density approaches infinity.
- Hence it cannot be used for predicting speed at high density.
- However it shows better fit than green shield and free field for uncongested traffic.

$$\text{Also, } q = kv = k v_f e^{-k/k_0}$$

$$\text{For } q_{\max}, \frac{dq}{dk} = 0.$$

$$\frac{dq}{dk} = k v_f e^{-k/k_0} \left(-\frac{1}{k_0}\right) + v_f e^{-k/k_0}$$

$$\frac{k}{k_0} = 1$$

$$k = k_0$$

$$\text{and for } q = q_{\max}, v = v_0$$

$$v_0 = v_f e^{-k_0/k_0}$$

$$v_0 = v_f / e$$

$$q_{\max} = k_0 v_0 = \frac{k_j v_f}{2e}$$

$$k_0 = \frac{k_j}{2}$$

(iv) PIPE'S GENERALIZED MODEL :-

- In order to generalize the modelling approach pipe introduced a new parameter "n".
- He proposed a model represented by following eqn

$$v = v_f \left[1 - \left(\frac{k}{k_j} \right)^n \right]$$

Note:- For $n=1$, This becomes GREENSHIELD MODEL.

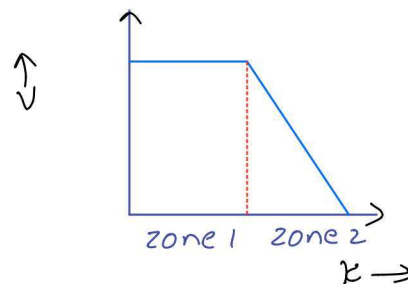
- Thus by varying "n", families of model can be developed.

(v) MULTI-REGIME MODEL :-

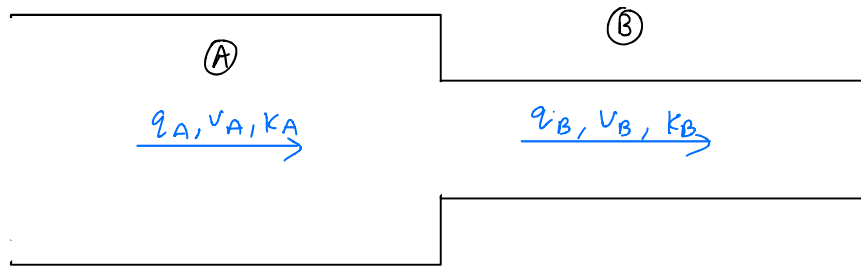
- All the above models are based on the assumption that the same speed-density relation is valid for the entire range of densities in traffic stream.
- Hence these models are termed as single regime model.
- However human behaviour will be different at diff. densities.
- Hence speed-density relationship will also be different in diff. zones of densities.
- Based on this many models were proposed, termed as **multi regime models**. Eg:- Two regime model.

zone-I :- $q = kv$ $\rightarrow$ const
Relation b/w q & v is linear

zone II :- $q = kv$
 $= k(k_c)$
 $= k^2 c \rightarrow$ parabolic

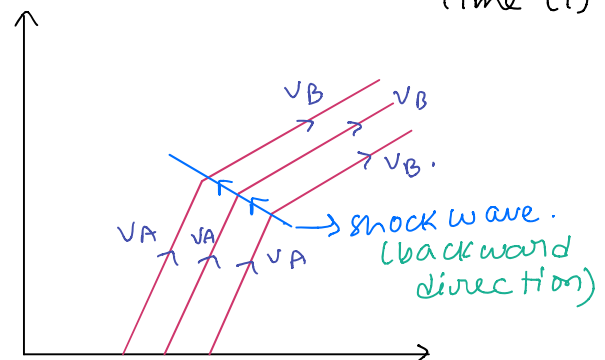
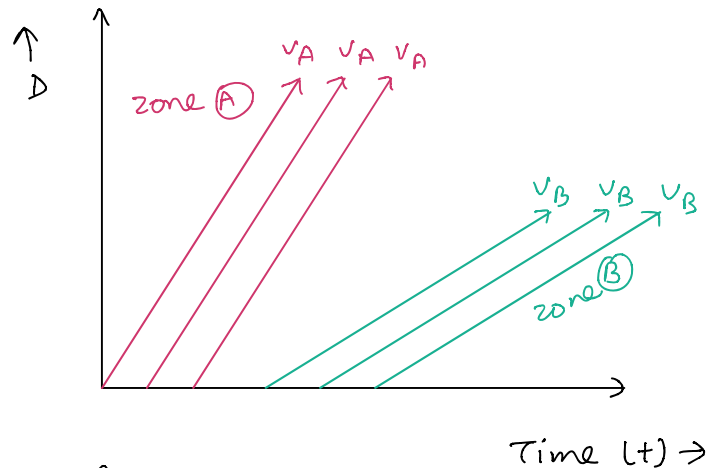
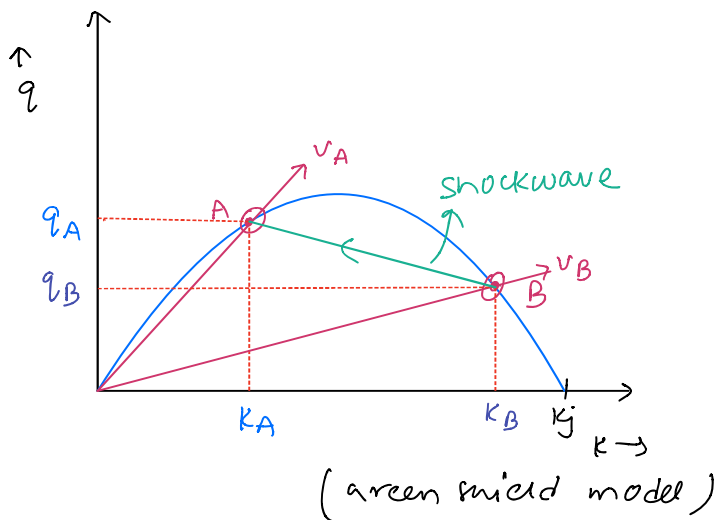


(vi) SHOCKWAVE MODEL :-



SHOCKWAVE MODEL

- The flow of traffic along a stream can be considered similar to a fluid flow.
- Consider a stream of traffic flowing with steady state condition i.e. all the vehicles in a stream are moving with constant speed, k , and q .
- Let suddenly due to some obstructions in the stream (eg, landslide, accident, vehicle failure, traffic block etc). the steady state characteristics changes and they acquire another state of flow.



- The speed of vehicle at state (A) is given by line OA and at stage "B" is given by line OB
- The same speed can also be shown on space time curve of traffic stream.
- All the lines are having the same slope which implies that they are moving with constant speed.
- The sudden change in the characteristics of the stream leads to formation of a shockwave. Hence shockwave is movement of the point that differentiate the two stream conditions.
- The slope of line AB gives the speed of shockwave.

$$w_{AB} = \frac{q_A - q_B}{K_A - K_B}$$

NOTE :- The above relationship can also be computed analytically, by conserving no. of vehicles passing through section A & B.

$$N_A = N_B$$

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$$N_A = q_A \times t$$

$$N_B = q_B \times t$$

$$N_A = K_A (v_A - w_{AB})$$

$$N_B = K_B (v_B - w_{AB}) t$$

$$\text{But } N_A = N_B$$

$$K_A (v_A - w_{AB}) t = K_B (v_B - w_{AB}) t$$

$$K_A v_A - w_{AB} (K_A - K_B) = K_B v_B$$

$$q_A - q_B = w_{AB} (K_A - K_B)$$

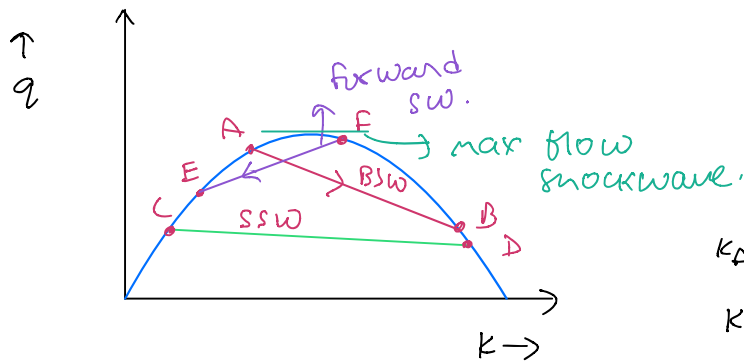
$$w_{AB} = \frac{q_A - q_B}{K_A - K_B}$$

- Shockwave can move against the direction of traffic, this type of shockwave is termed as backward moving shockwave

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Shockwaves can also be forward moving if it moves along with the traffic direction.
(Eg. width of section increases suddenly or signal changes from red to green $\rightarrow$ velocity increases).

- Stationary shockwave will occur when two streams having same flow value but diff densities meet



$$K_A < K_B \rightarrow q_A > q_B$$

$$K_E < K_F \rightarrow q_E < q_F$$

$$K_C = K_D \rightarrow q_C = q_D$$

Jan 26 (II) MICROSCOPIC MODEL :-

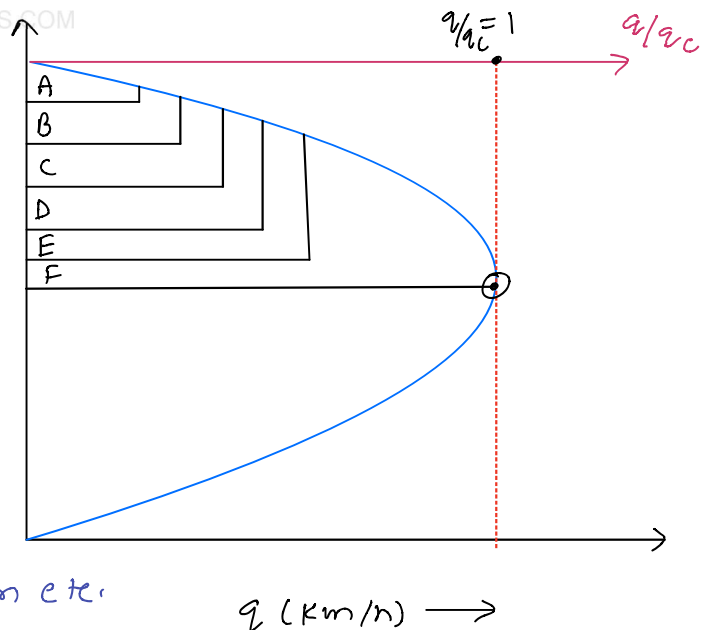
- In this model of traffic flow, analysis of flow of traffic is done by modelling driver to driver and driver to road interaction with in a traffic stream which respectively analyse the interaction b/w a driver and another driver on the road.

LEVEL OF SERVICE (LOS)

- LOS is qualitative measure used to relate the quality of motor vehicle traffic service.

↑
v
(kmph)

- LOS is used to analyse roadways & intersections by categorizing traffic flow & assigning quality levels of traffic based on performance measure like speed, density, congestion etc.



NOTE:- Here quantitative measure of flow is given by traffic capacity.

- Factors to be considered for the evaluation of LOS of a roadway includes

- (a) operating speed.
- (b) Travel time.

- (c) Traffic interruptions.
- (d) Freedom of movement.
- (e) Driving comfort.
- (f) Safety.
- (g) Economy.

- As per HCM (Highway capacity manual) following factors are considered to evaluate LOS.

- (a) operating speed.
- (b) Ratio of volume to capacity. (v/c)

- The HCM & IRC suggest six LOS (level of services) of a roadway ranging from A to F having following properties.

(i) LOS-A :-

- Free flow operation
- No restriction in maneuvering.
- low traffic
- High speed

(ii) LOS-B :-

- Reasonably free flow
- Ability to maneuver is slightly restricted.
- Effect of minor incidents still easily absorbed.
- Stable flow & noticeable traffic.

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(iii) LOS-C

- Speed is reduced in this case
- Freedom to maneuver is noticeably restricted
- Queues may be formed behind any significant blockage.
- Flow is stable.
- Significant traffic interaction takes place.

(iv) LOS-D :-

- Unstable flow condition.
- High density.
- Movement restriction.
- Speed decline significantly.
- Minor incident create queue

NOTE:- As per IRC, for design of rural roads LOS B is adopted and for design of urban roads LOS-C is adopted.

(v) LOS-E :-

- Unstable flow.
- lower speed
- volume $\approx$ capacity.
- very little freedom of movement.
- Any disruption causes queuing.

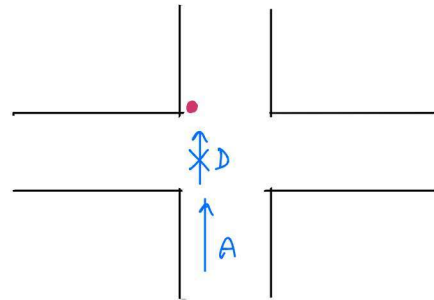
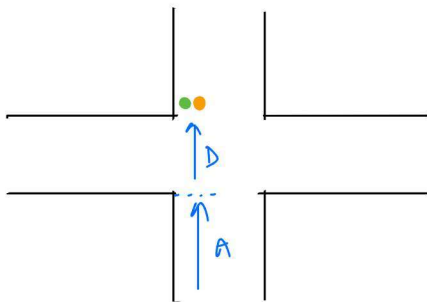
(vi) LOS-F :-

- Breakdown in flow.
- Queues from behind breakdown points.
- Here demand > capacity.

DELAY & QUEUE ANALYSIS AT SIGNALISED INTERSECTION

For green & Amber both arrival & departure are possible
Effective green time

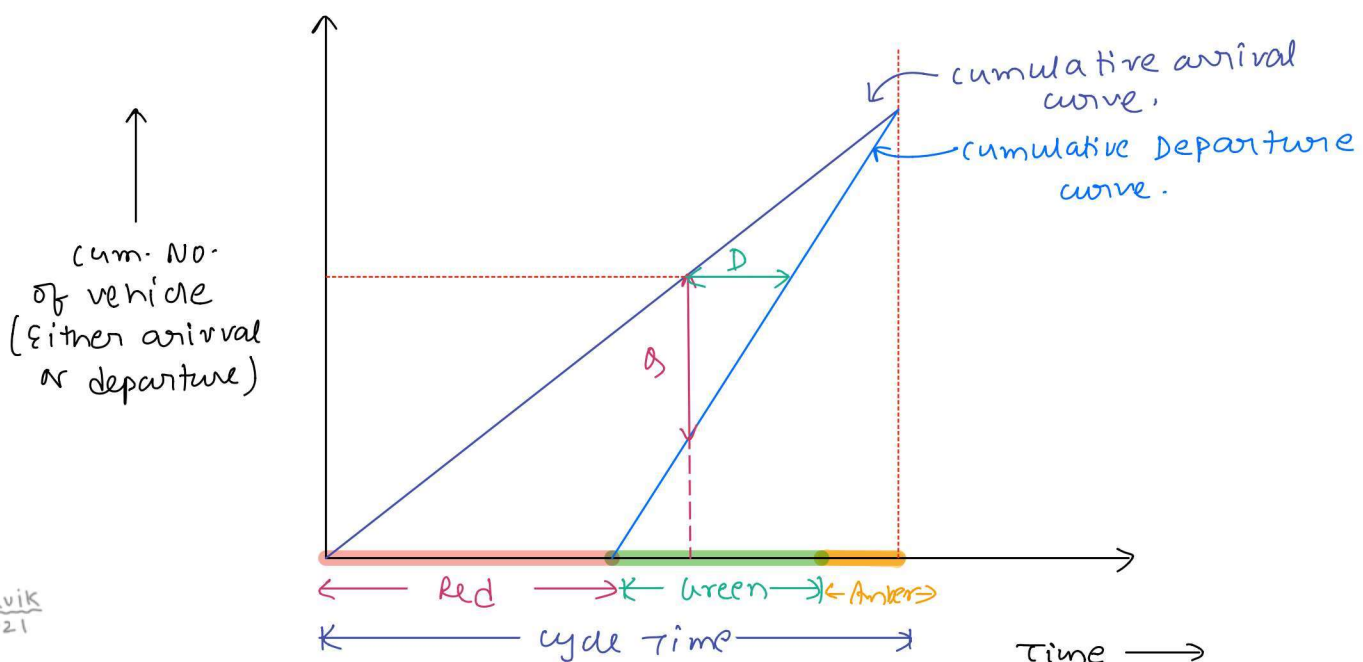
RED → Arrival but no departure



① STOP-TIME DELAY (D):- It is defined as the time a vehicle is stopped in queue while waiting to pass through signalized intersections.

- It begins when the vehicle is fully stopped and ends when the vehicle begins to accelerate.
- Avg. stopped time delay is the avg for all vehicles during the specified time period.
- It is mathematically diff of departure time & arrival time.

(ii) QUEUE (Q):- It is the diff of total number of vehicle arrival and total number of vehicle departure at any instant of time.



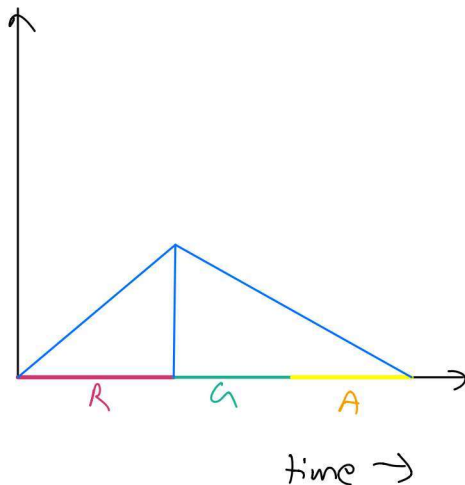
NOTE :- Here avg. stop time delay =
$$\frac{\left[\text{Area b/w cum. arrival \& cum. departure curve for one cycle} \right]}{\text{cum. no. of vehicle Arrival per cycle time}}$$

Assumptions made in previous analysis :-

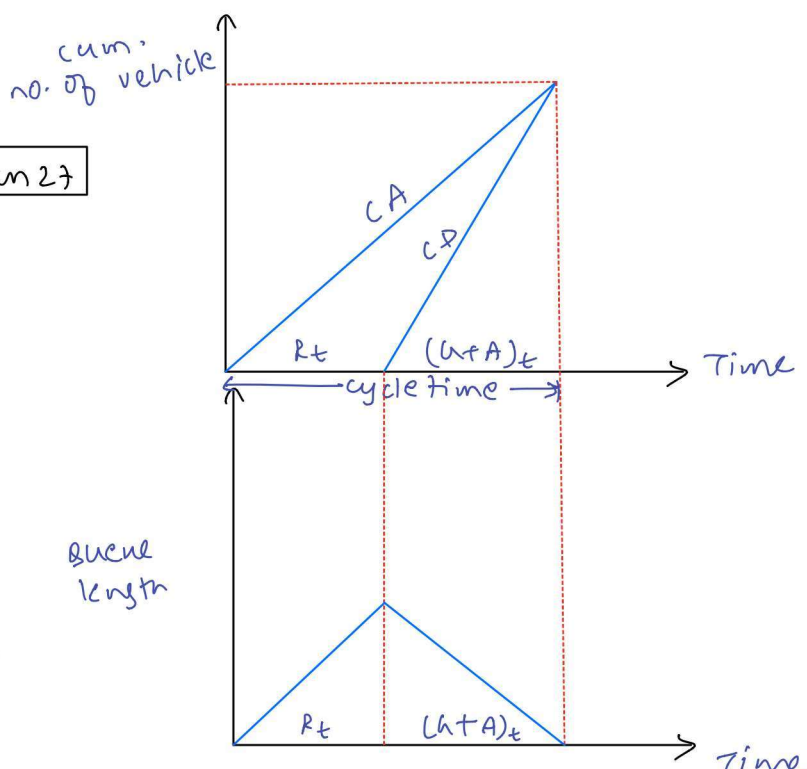
- (i) vehicles arrive and depart at uniform rate.
(Hence cum Arriv & Dept curve is linear with time).
- (ii) No pre-existing queue is there at intersection.
- (iii) Arriving vehicle departs instantaneously when the section
- (iv) The departure of the vehicles takes place at it max rate termed as SATURATION FLOW RATE/SERVICE RATE.
- (v) The departure curve catches up with the arrival curve before the next red interval begins.

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↑
queue
length



Jan 27

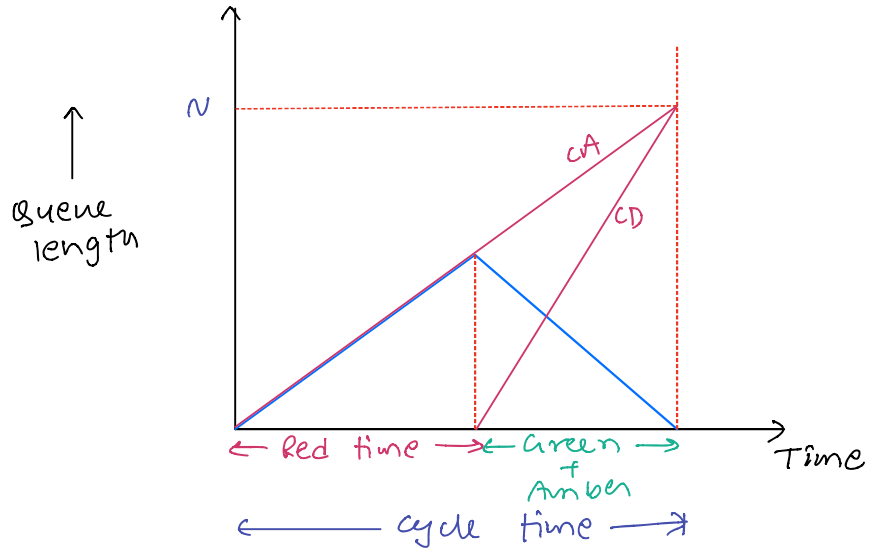


queue
length

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NOTE:- 1) Queue length v/s time graph can be converted into cum. arrival & departure v/s time graph.

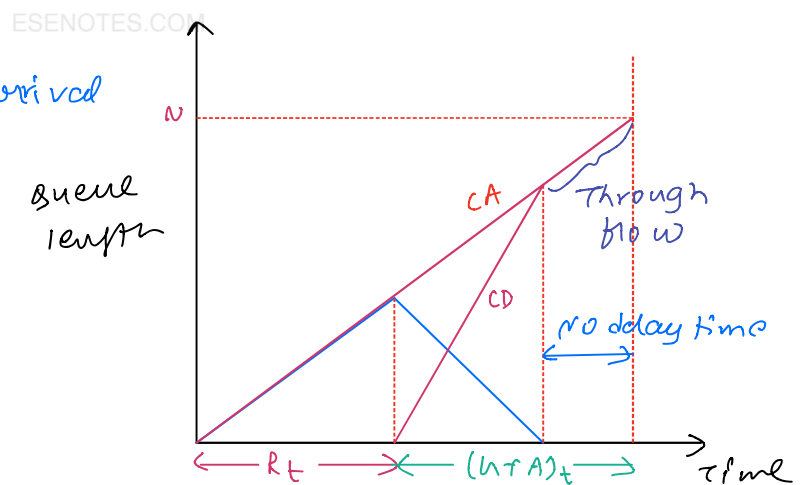
case (i) :- cum. departure matches cum. arrival exactly at the end of one cycle.



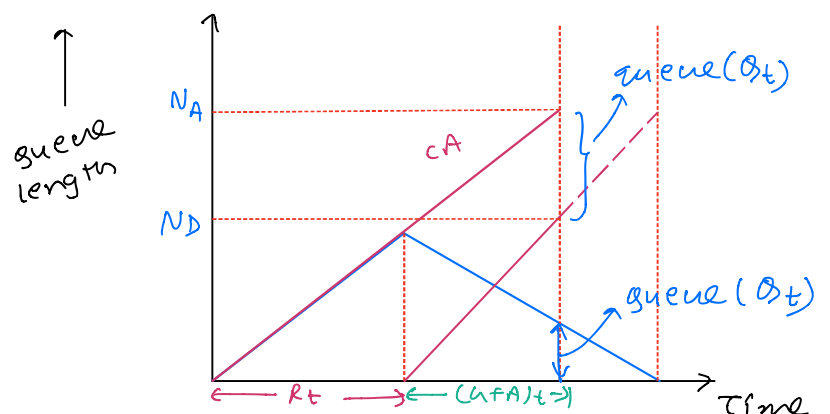
case (ii) cumulative departure matches cumulative arrival before the completion of cycle time.

→ cum. dep. curve meets arrival before cycle length.

→ Hence no delay time is there.



case (iii) cumulative departure doesn't matches cum. arrival even at the end of one cycle time.



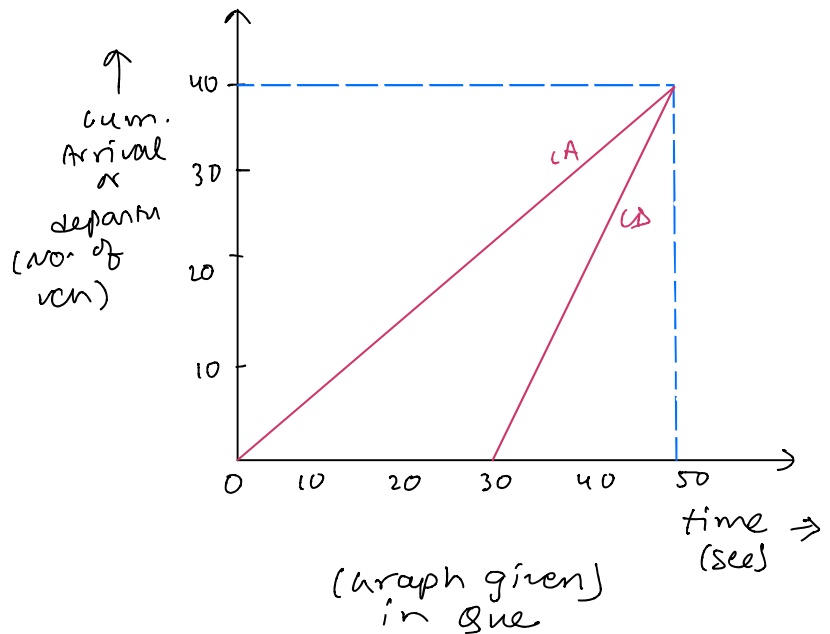
- 8) The cumulative arrival and departure curve of one cycle of an approach lane of a signalised intersection is shown. The cycle time is 50 sec and the effective red time is 30 sec, effective green time is 20 sec. what is average delay.

Sol cycle time = 50 sec.

$$R_t = 30 \text{ Sec}$$

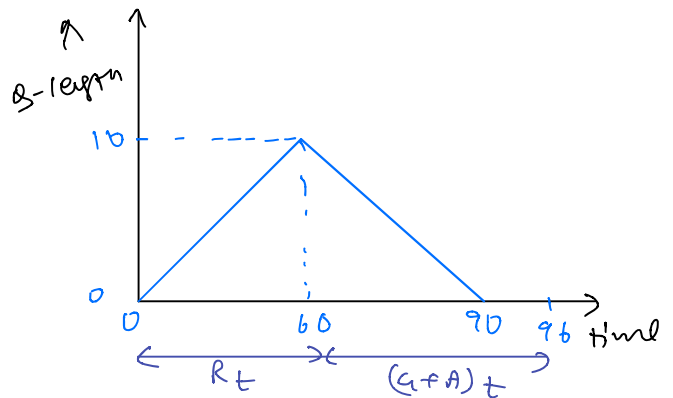
$$G_t = 20 \text{ sec}$$

$$\begin{aligned} \text{Avg delay} &= \frac{\text{Area under curve}}{\text{no. of arrived vehicle}} \\ &= \frac{\frac{1}{2} \times 30 \times 40}{40} = 15 \text{ sec} \end{aligned}$$



- 9) The queue length in number of vehicles vs time (sec) plot for an approach to a signalised intersection with the cycle length of 96 sec is shown.

- At time $t=0$, the light has just turned red, the effective green time is 36-sec, during which vehicles discharge at saturation flow rate s (veh/hr). vehicle arrive at a uniform rate v (veh/hr) throughout the cycle. compute s , v , and avg stopped delay per veh.



Sol $\triangle ABG \sim \triangle AEF$

$$\frac{10}{EF} = \frac{60}{96}$$

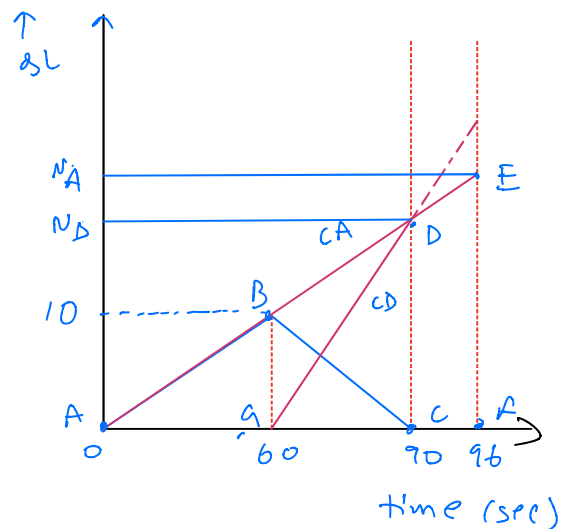
$$EG = 16$$

$$\triangle ABG \sim \triangle ADC$$

$$\frac{10}{DC} = \frac{60}{90}$$

$$DC = 15$$

$$\text{Avg delay} = \frac{\frac{1}{2} \times 60 \times 15}{16} = 28.125 \text{ sec.}$$



Rate of departure = slope of CD, $s = \frac{CD}{t_c} = \frac{15}{30} \times 3600 = 1800 \text{ uph}$

Rate of arrival = slope of AD, $v = \frac{DC}{t_a} = \frac{15}{90} \times 3600 = 600 \text{ uph}$

8) For a particular traffic signalised intersection, uniform arrival rate is 18 veh/min and uniform depart rate is 25 veh/min.

compute.

- (i) Avg delay per vehicle.
- (ii) length of queue at end of cycle.

Total cycle time = 60 sec, $R_t = 30 \text{ sec}$ & $E_t = 30 \text{ sec}$

(iii) In next cycle time, uniform arrival rate is same but uniform dept. rate changes to 52 veh/min. Then find

- (a) Avg delay per vehicle.
- (b) Total no. of vehicles which will not have any delay.
- (c) length of the longest queue.

Sol

Arrival rate = 18 veh/min.
dept. rate = 25 veh/min.

(i) Avg delay = $\frac{\text{Area under curve}}{N_A}$ am. veh.

$$= \frac{\frac{1}{2} \times 60 \times 18 - \frac{1}{2} \times 30 \times 12.5}{18}$$

$$= 19.58 \text{ sec.}$$

(ii) length of Q = $18 - 12.5$
= 5.5 veh.

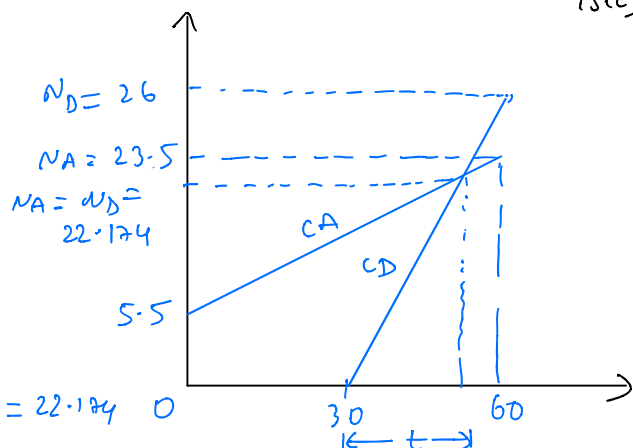
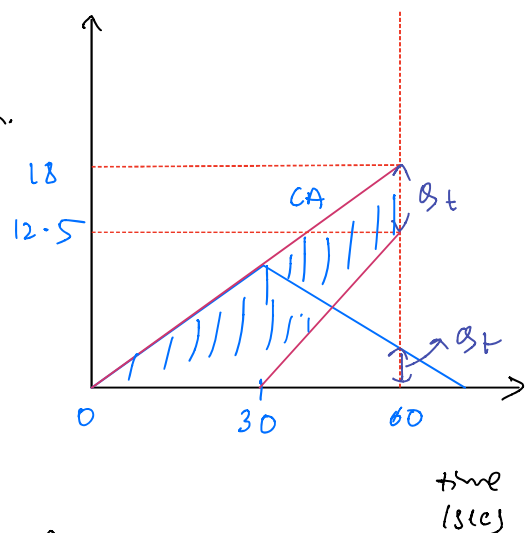
(iii) Next cycle time

Arr. rate = 18 veh/min
Dept. rate = 52 veh/min.

(a) $5.5 + 18 \times \frac{(30+t)}{60} = 52 \times \frac{t}{60}$

$$t = 25.58 \text{ sec}$$

$N_A = N_D = 5.5 + 18 \times \frac{(30 + 25.58)}{60} = 22.174$



$$\text{Avg delay} = \frac{\frac{1}{2} \times (22.17 + 5.5) \times 55.58 - \frac{1}{2} \times 25.58 \times 22.174}{23.5}$$

$$= 20.65 \text{ sec.}$$

(b) no. of vehicle with no delay = $23.5 - 22.174$
 $= 1.326$

(c) length of longest queue = $5.5 + 18 \times \frac{30}{60} = 14.5 \text{ veh.}$

PROBABILITY OF VEHICLE ARRIVAL :-

- Arrival of vehicle at a particular section of road is considered to follow "poisson's distribution."
- Probability that "N" no of vehicles will arrive within "t" sec time interval.

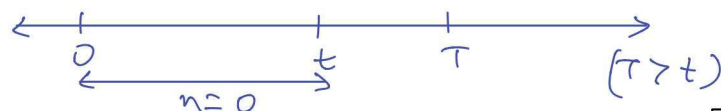
$$P(n, t) = \frac{(\lambda t)^n e^{-\lambda t}}{n!}$$

λ = vehicle arrival rate

t = time interval or time headway.

n = no. of vehicle.

case (i) :- Probability that vehicle will arrive after time "t".



probability that time headway $\geq t$, $P = e^{-\lambda t}$

case (ii) :- probability that veh. arrive within time "t".

P of vehicles arriving within time t + P of vehicle arriving t = 1

P of vehicle arriving within time t = 1 - P of vehicle arriving after time "t"

$$P = 1 - e^{-\lambda t}$$

Jan 28 (iv) TRAFFIC SPEED STUDIES

- The speed of the different vehicles over a particular route depends on several factors such as.

- (i) Geometric design of road.
- (ii) Driver characteristics.
- (iii) climate & environmental factors.
- (iv) Time of travel.
- (v) Traffic volume.
- (vi) Purpose of travel.

NOTE:- 1) Spot speed: It is the instantaneous speed of a vehicle at a specified cross-section or location.

- The spot speed of a vehicle fluctuates from place to place all along the routes depending upon the above factors.

2) Average speed: It is the average of spot speed of all the vehicles passing a given point on highway.

3) Running speed: It is avg speed maintained by a vehicle over a particular stretch of road while the vehicle is in motion.

$$\text{Running speed} = \frac{\text{Journey length}}{\text{Running time}}$$

$$\text{Running time} = \text{Journey time} - \text{delay}$$

4) Travel speed: It is avg speed of travel for a vehicle over a particular stretch of road, for entire travel time.

$$\text{Travel speed} = \frac{\text{Journey length}}{\text{Journey time}}$$

- Traffic speed studies are generally of following types.

- (a) SPOT SPEED STUDIES.
- (b) SPEED & DELAY STUDIES.

(a) SPOT SPEED STUDIES :-

- It is referred as instantaneous speed of the vehicle at a point / cross-section on road.
- Measurement of spot speed can be done by any of following methods.

(i) By noting the time taken by vehicle to travel a known distance of 15 or 30m either manually or mechanically, or automatically.

Hence speed is given by $v = d/t$ (m/sec).

(ii) Spot speed can also be measured with the help of several equipments, like

- Graphic recorder.
- Electronic meter.
- Photo electronic meter.
- Photographic meter.
- Radar speed meter.
- Enoscope.

- Spot speed studies are being carried out for following purpose

- for planning of traffic regulations.
- Design or Re-design of geometric elements.
- In accident studies & preventive measures.
- To find the speed trend over the period of time.

- Depending upon the method of measurement of spot speed i.e. over a particular stretch of road or by equipment different types of avg speed is calculated.

(i) SPACE-MEAN SPEED :- It represents the avg speed of the vehicle in a certain road length at any time.

- This is obtained from the observed travel time of the vehicles over a stretch of road.

$$v_s = \frac{n}{\frac{1}{v_1} + \frac{1}{v_2} + \frac{1}{v_3} + \dots + \frac{1}{v_n}}$$

$$v_i = \frac{d}{t_i} \rightarrow \text{fixed (15-30)m}$$

↳ It is harmonic mean of diff. spot speed.

$$V_s = \frac{n}{\frac{t_1}{d} + \frac{t_2}{d} + \dots + \frac{t_n}{d}} = \frac{dn}{t_1 + t_2 + \dots + t_n}$$

$$V_s = \frac{dn}{\sum t_i \rightarrow \text{sec}} \quad (\text{m/s})$$

$$V_s = 3.6 \frac{dn}{\sum t_i} \quad (\text{km/hr}).$$

V or V_s = space mean speed

d = length of road

n = no. of individual veh. observation.

t_i = observed time

NOTE:- The average travel time of all the vehicles is obtained from reciprocal of space mean speed.

$$t_a = \frac{1}{V_s} = \frac{\sum t_i}{dn}$$

(ii) TIME MEAN SPEED :-

- It represents the speed distribution of vehicles at a point on the roadway & it is avg of instant speed of observed vehicles at a spot.

$$V_t = \frac{V_1 + V_2 + V_3 + \dots + V_n}{n}$$

$$V_t = \sum \frac{V_i}{n} \quad (\text{m/s})$$

$$V_t = 3.6 \frac{\sum V_i}{n} \quad (\text{kmph})$$

→ Arithmetic mean of spot speed.

V_t, V_s → Time mean speed.

V_i = observed inst. speed of i th vehicle.

n = no. of vehicle observed.

Presentation of Spot Speed data :-

It can be done by following.

i) speed distribution Table.

- From spot speed data frequency distribution table of spot speed can be prepared, by arranging the speed groups covering desired speed range and the number of vehicles in each speed range.

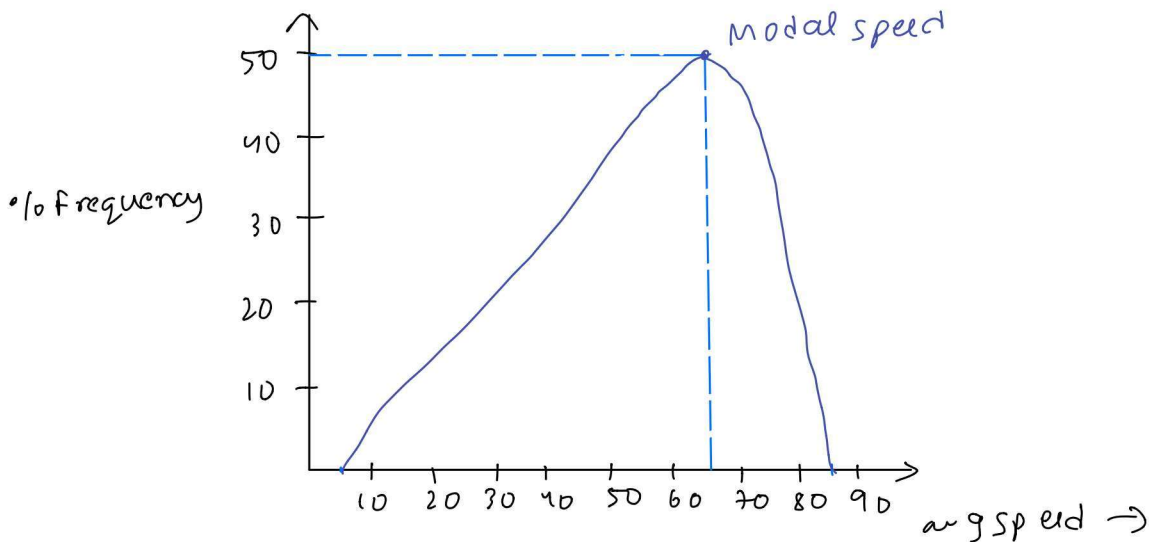
Eg:-

Speed range (kmph)	No. of speed observed / frequency	% frequency
0-10	2	$\frac{2}{400} \times 100 = 0.5$
10-20	5	1.25
20-30	10	2.5
30-40	15	3.75
40-50	30	7.5
50-60	55	13.75
60-70	200 $\Rightarrow$ max	50
70-80	63	15.75
80-90	20	5
	<u>400</u>	

(i) Frequency distribution curve :-

- A graph is plotted with the average values of each speed group of vehicles on the x-axis and the % of vehicles in that group on y-axis.
- This curve is termed as Frequency distribution curve.
- This curve has a definite peak of travel speed across the section & this speed is denoted as modal speed.

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(iii) Cum. freq. distribution curve :-

- It is the curve plotted with avg values of each speed group on y-axis
- with the help of this curve following significant speed are noted.

(a) 98th Percentile speed :-

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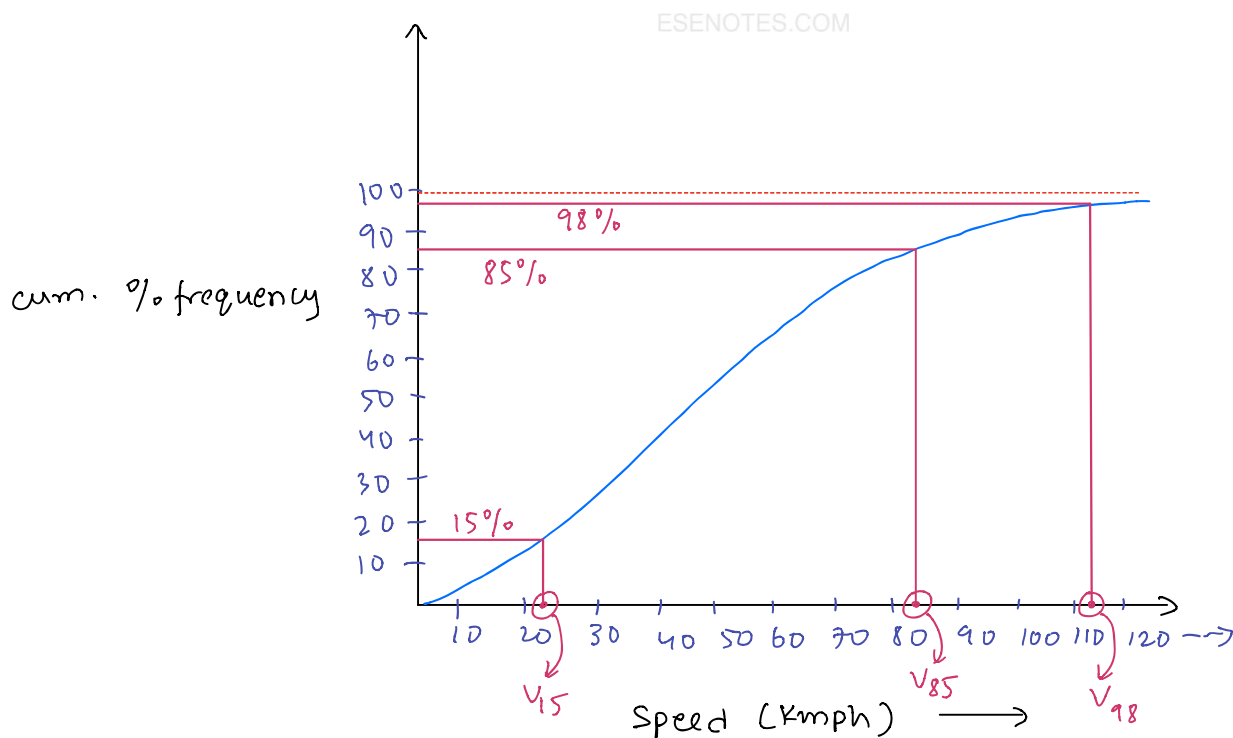
- It is the speed at or below which 98% of the vehicles are moving and only 2% vehicles exceeds this limit.
- As per IRC, this speed is considered as design speed for geometric elements.

(b) 85th percentile speed :-

- It is the speed at or below which 85% vehicles are moving and only 15% of vehicle exceeds this limit.
- The vehicles exceeding this limit are considered to be driving faster than safe speed.
- Hence it is considered as upper safe limit.

(c) 15th percentile speed :-

- For the purpose of avoiding congestion/Jam over a particular road this speed is determined as it is considered as lower safe limit.



(iv) SPEED DISPERSION :-

- The speed distribution of vehicles generally follow normal distribution curve.
- The amount of spread of speed from avg speed is termed as dispersion, which can be reported in terms of following.
 - (a) 85th - 15th percentile speed.
 - (b) Standard deviation of spot speed.
 - (c) Coeff of variation of spot speed.

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- 8) Spot speed studies were carried out at a certain stretch of a highway with mixed traffic flow and the consolidated data collected are given below.

Speed range (kmph)	No. of vehicles observed	Speed range (kmph)	No. of vehicles observed.
0 - 10	12	60 - 70	119
10 - 20	18	70 - 80	43
20 - 30	68	80 - 90	33
30 - 40	89	90 - 100	9
40 - 50	204		
50 - 60	255		

Determine

- The upper and lower values of speed limit for installing speed regulation.
- The design speed for checking the geometric design elements of the highway.

Sol i) upper speed limit $\rightarrow$ 85th %ile
lower " " $\rightarrow$ 15th %ile

Speed Range	Avg Speed	No. of vehicle	% freq.	cum % freq
0 - 10	5	12	1.41	1.41
10 - 20	15	18	2.12	3.53
20 - 30	25	68	8	11.53
30 - 40	35	89	10.47	22
40 - 50	45	204	24	46
50 - 60	55	255	30	76

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60-70	65	119	14	90
70-80	75	43	5.06	95
80-90	85	33	3.88	98
90-100	95	9	1.06	100
		<u>Σ = 850</u>		

85th %ile speed = 61.5 kmph
 15th %ile speed = 28.31 kmph.

98th %ile speed = 85 kmph.

(b) SPEED & DELAY STUDIES :-

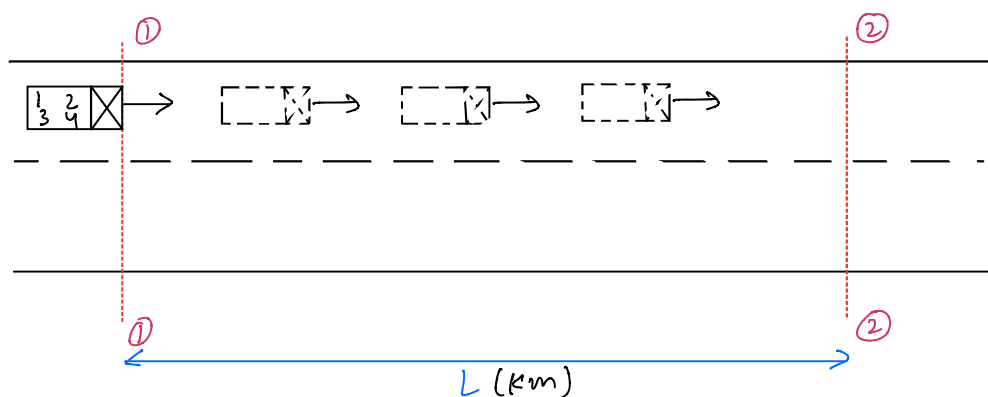
- This study gives the idea of running speed or the fluctuations in speed, the location and duration of delays or stoppage and the overall travel speed b/w two desired location along the road.
- The result of this study can further be used in identifying the location of congestion, causes of delay hence helps in improvement of existing road or in design of new road.
- The delay is further classified into two.

(a) Fixed delay :- These occur particularly at intersection due to traffic signals.

(b) Operational delay :- These occurs due to interference of traffic movements such as turning of vehicle, parking & unparking of vehicles or pedestrian etc.

- These studies are being carried out by several methods :-

(a) FLOATING CAR OR RIDING CHECK METHOD :-



NOTE:- This method is applicable for two lane two way road.

- In the floating car method, a test vehicle is driven over a given route to travel at approx the avg speed of the stream, thus trying to "float" with the traffic stream.
- The no. of test run are made along the study stretch
- A group of 4- observer are seated in the test vehicle to record various observation during the test in each run as follows :-

OBSERVER ① :- It is provided with two stopwatches from which.
 ONE is used to record the total journey time
 OTHER is used to record the delay time at different locations.

OBSERVER ② :- It notes the time, location and cause of the delays during each test run

OBSERVER ③ :- It notes no. of vehicles overtaking the test vehicle and no of vehicles overtaken by test vehicle.

OBSERVER ④ :- It notes the number of vehicles travelling in the opposite direction in each test run.

According to this method following results are obtained

Traffic flow :-

$$q = \frac{\eta_a + \eta_f}{t_w + t_a}$$

Journey time of flow :-

$$\bar{T} = t_w - \frac{\eta_f}{q}$$

t_w = avg journey time of test vehicle when running / moving with the flow.

t_a = avg journey time of test vehicle when running against the flow.

can be +ve, -ve or even zero. $\eta_f = \left[\begin{array}{l} \text{avg no. of vehicles} \\ \text{overtaking the test} \\ \text{vehicle} \end{array} \right] - \left[\begin{array}{l} \text{avg no. of vehicles} \\ \text{overtaken by} \\ \text{test vehicle} \end{array} \right]$

n_a = avg no. of vehicles on study stretch (L km) in the direction of flow.

[This data is recorded by 4th observer while travelling in opposite direction.]

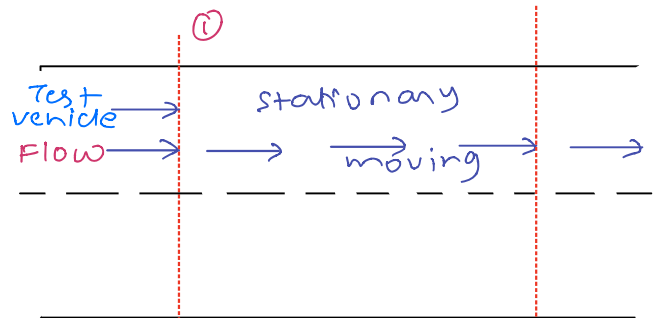
- The above relationship is being derived as follows.

(i) Test vehicle is at rest and flow is moving.

$$\text{Flow, } q = \frac{n_o}{T}$$

$$n_o = qT$$

↳ overtaking vehicle.



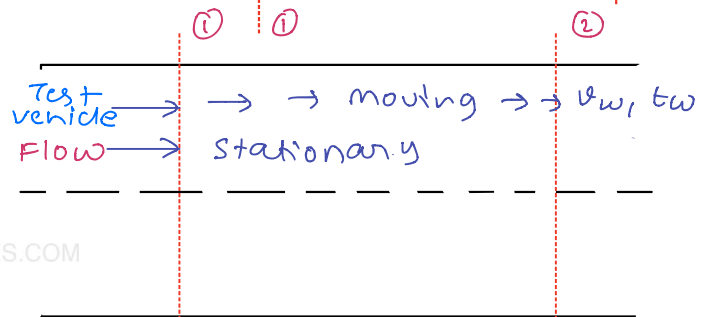
(ii) Test vehicle is moving but flow is at rest

→ Avg. Speed of
 $L = v_w t_w$ test vehicle.

$$K = \frac{n_s}{L} \rightarrow \text{no. of overtaken vehicle.}$$

$$n_s = KL$$

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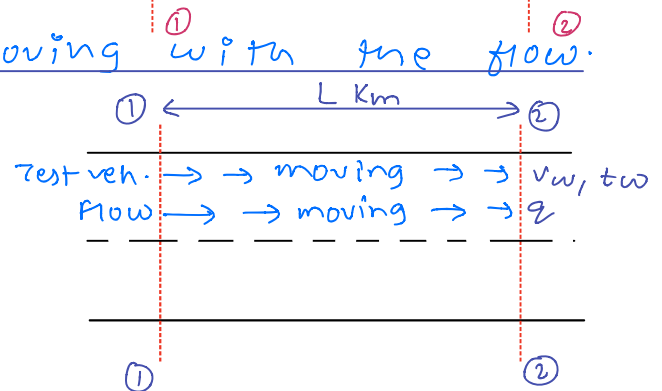
(iii) when test vehicle is moving with the flow.

$$n_o = q t_w$$

$$n_s = KL = K v_w t_w$$

$$n_r = n_o - n_s$$

$$\left. \begin{aligned} n_r &= q t_w - K v_w t_w \\ n_r &= q t_w - KL \end{aligned} \right\} - (i)$$



(iv) when test vehicle is moving against the flow. :-

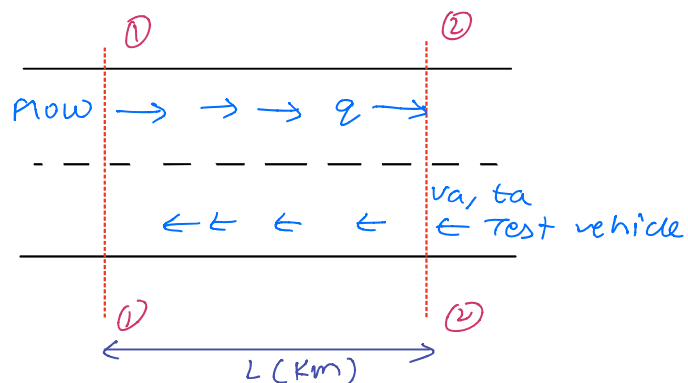
$$n_a = n_o - n_s$$

$$= q t_a - KL$$

$$= q t_a - K (-v_a \cdot t_a)$$

$$n_a = q t_a + K v_a t_a \left\} - (ii)\right.$$

$$\therefore n_a = q t_a + KL$$



From (i) & (ii)

$$\begin{aligned} n_r &= q t_w - K L \\ n_a &= q t_a + K L \\ \hline n_r + n_a &= q(t_w + t_a) \end{aligned}$$

$$q = \frac{n_r + n_a}{t_a + t_w}$$

From eqn (i)

$$\begin{aligned} n_r &= q t_w - K L = q t_w - \frac{q}{v} \cdot v \bar{t} \\ n_r &= q t_w - q \bar{t} \end{aligned}$$

$$\bar{t} = \frac{q t_w - n_r}{q}$$

- 8) The consolidated data collected from speed and delay studies by floating car mtd on a stretch of urban road of length 3.5 km, running N-S are given. Determine the avg. value of
 (i) volume (ii) Journey speed (iii) Running speed of the traffic stream along each direction.

Trip No	Direction of Trip	Journey Time (min-sec)	Total stopped delay (min-sec)	No. of vehicles overtaking	No. of vehicle overtaken	No. of veh. from opposite direct
1	N-S	6-32	1-40	4	7	268
2	S-N	7-14	1-50	5	3	186
3	N-S	6-50	1-30	5	3	280
4	S-N	7-40	2-00	2	1	200
5	N-S	6-10	1-10	3	5	250
6	S-N	8-00	2-22	2	2	170
7	N-S	6-28	1-40	2	5	290
8	S-N	7-20	1-40	3	2	160

Direction N-S	Journey Time	stopped delay time	no. of vehicle		In opp. direction
	6-32	1-40	4	7	268
	6-50	1-30	5	3	280
	6-10	1-10	3	5	250
	6-28	1-40	2	5	290
Total -	26-00	6-00	14	20	1088
Arg -	6-30	1-30	3.5	5	272

Direction S-N	Journey time	Stop delay time	no. of vehicles		
			Overtaking	overtaken	gn opp dir.
	7-14	1-50	5	3	186
	7-40	2-00	2	1	200
	8-00	2-22	2	2	170
	7-20	1-40	3	2	160
Total	30-14	7-56	12	8	716
avg	7-33	1-58	3	2	179

N-S dir.

$$n_r = \text{overtaking} - \text{overtaken}$$

$$= 3 - 5 = -1.5$$

$$n_a = 179$$

$$t_w = 6.5 \text{ min.} \quad t_a = 7.55$$

$$q = \frac{n_a + n_r}{t_w + t_a} = \frac{-1.5 + 179}{(6.5 + 7.55)} = 12.63 \text{ (veh/min)}$$

$$\bar{t} = t_w - \frac{n_r}{q} = 6.5 + \frac{1.5}{12.63} = 6.62 \text{ (min)}$$

$$\text{Journey speed, } = \frac{L}{\bar{t}} = \frac{3.5 \times 60}{6.62} = 31.72 \text{ kmph.}$$

$$\text{Avg running time} = 6.62 - 1.5 = 5.12 \text{ min.}$$

$$\text{Avg running speed} = \frac{3.5 \times 60}{5.12} = 41 \text{ kmph.}$$

S-N direction

$$n_r = \text{overtaking} - \text{overtaken}$$

$$= 3 - 2 = 1$$

$$n_a = 272, \quad t_w = 7.55 \text{ min}, \quad t_a = 6.5 \text{ min}$$

$$q = \frac{n_r + n_a}{t_w + t_a} = \frac{1 + 272}{7.55 + 6.5} = 19.43 \text{ (veh/min)}$$

$$\text{Avg journey time } \bar{t} = t_w - \frac{n_r}{q} = 7.55 - \frac{1}{19.43} = 7.6 \text{ min}$$

$$\text{Journey speed} = \frac{3.5 \times 60}{7.6} = 27.63 \text{ (kmph)}$$

$$\text{Running time} = 7.6 - 1.96 = 5.64 \text{ min.}$$

$$\text{avg running speed} = \frac{3.5 \times 60}{5.64} = 37.4 \text{ (kmph)}$$

Jan 30

8) An observer travelling with a constant speed of 60 kmph with the traffic stream over 4 km stretch is passed by 20 vehicles more than what he passes. When the observer travels against the stream at the same speed, number of vehicles he meets is 330. Find the number of vehicles in the direction of flow.

sol

$$n_r = \text{over taken} - \text{overtaking} = 20$$

$$n_a = 330$$

$$t_w = t_a = \frac{4}{60} \times 60 = 4 \text{ min.}$$

$$q = \frac{n_r + n_a}{t_w + t_a} = \frac{20 + 330}{8} = 43.75 \text{ (veh/min)}$$

**

8) A student driving a bicycle on a 5 km one way street takes 40 min to reach home. The student stopped for 15 min during the ride. 60 vehicles overtook the student (Assume the no. of veh. over taken by student is zero) during the ride. and 45 veh. while the student stopped. The speed of veh. stream in kmph on that road is.

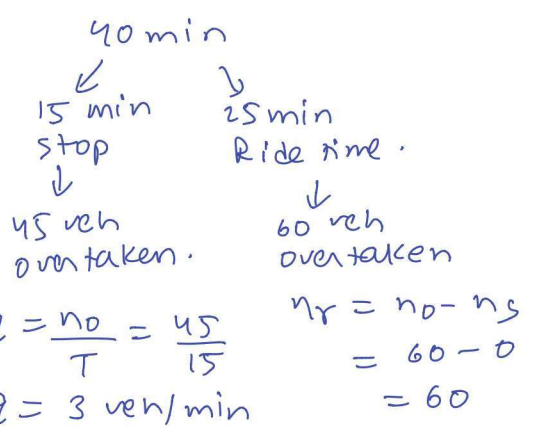
sol

$$\text{vehicle stream speed} = \frac{L}{\text{Journey time } (\bar{t})}$$

$$\bar{t} = t_w - \frac{n_r}{q}$$

$$\begin{aligned} \bar{t} &= 25 - \frac{60}{3} \\ &= 25 - 20 = 5 \text{ min} \end{aligned}$$

$$\text{vehicle speed} = \frac{5}{5} \frac{\text{km}}{\text{min}} = 1 \text{ (km/min)}$$



- Further from speed delay analysis result following can be concluded

$$\begin{aligned} n_r &= n_o - n_s \\ &= q t_w - K L \end{aligned}$$

$$n_r = q t_w - K v_w t_w \quad (i)$$

$$\frac{n_r}{t_w} = q - v_w K = K v - v_w K$$

→ v → vehicle
 → v_w → flow

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$$\underbrace{\left(\frac{n_r}{t_w}\right)}_{\text{Relative flow}} = \underbrace{K(v-v_w)}_{\text{Relative vel.}} \quad \text{--- (A)}$$

$$\therefore \text{Relative flow} = K \times \text{Relative velocity}$$

$$n_a = n_o - n_s$$

$$n_a = q t_a + K v_a t_a$$

$$n_a = K v t_a + K v_a t_a$$

$$n_a = K t_a (v + v_a)$$

$$\left(\frac{n_a}{t_a}\right)$$

$$= K (v + v_a)$$

Rel. velocity

Relative flow

$$\begin{array}{c} \leftarrow v_a \\ \rightarrow v \end{array}$$

$$\begin{aligned} \text{Rel. vel} &= v_a - (-v) \\ &= v_a + v \end{aligned}$$

$$\text{Relative flow} = \text{Density} \times \text{Relative speed}$$

- Q) while travelling along and against the traffic stream a moving observer measured the relative flow as 50 veh/hr & 200 veh/hr respectively. The avg speeds of the moving observer while travelling along and against the stream are 20 kmph & 30 kmph respectively. The density of the traffic stream in vehicles/km is.

sol $(\text{Rel. flow})_{\text{along}} = 50 \text{ veh/hr}$

$(\text{Rel. flow})_{\text{against}} = 200 \text{ veh/hr.}$

$$\frac{n_r}{t_w} = q - K v_w$$

$$50 = q - K \times 20 \quad \text{--- (i)}$$

$$\frac{n_a}{t_a} = q + K v_a$$

$$200 = q + K \times 30 \quad \text{--- (ii)}$$

$$400 = 2q + 60K$$

$$150 = 3q - 60K$$

$$5q = 550$$

$$q = 110 \text{ veh/hr.}$$

$$K = 3 \text{ veh/km.}$$

(b) LICENCE PLATE METHOD :-

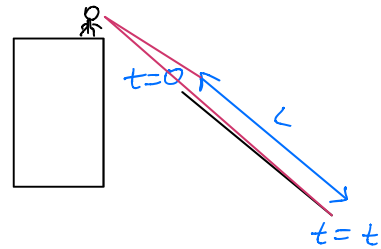
- In this method, observers are stationed at the entrance & entry of the test stretch where the information of travel time is required.
- The timing of vehicle entering and leaving the test stretch is noted to compute the vehicle speed.

(c) INTERVIEW TECHNIQUE :-

→ interview of rider.

(d) Elevated observation & photographic Techⁿ.

- Chhat pai charke dekhte hai.



ORIGIN & DESTINATION STUDIES :-

- O & D studies gives the information on the actual location or zone of origin of travel of vehicles & their destinations.
- These studies provide details such as direction of travel, section of routes, trip length, frequency & number of such trip and no. of individuals travelling in particular trip.
- These studies find their application in.

(a) To judge the adequacy of existing routes.

(b) To establish design standards.

(c) To locate expressway & major roads.

(d) To locate new bridge as per traffic demand.

(e) To identify potential congestion points.

- O & D studies can be carried out by any of the following methods :-

(a) ROAD SIDE INTERVIEW METHOD :- interview at the road.
• congestion, wrong information.

(b) License plate method :- same as

(c) Return post card method :- questions will be present in post card.
- Inaccurate information.

(d) Tag on card method :- fastag., origin & destination can be analysed
- costly

(e) Home interview mtd :- interviews at home.

- origin and destination studies can be present in any of following forms.

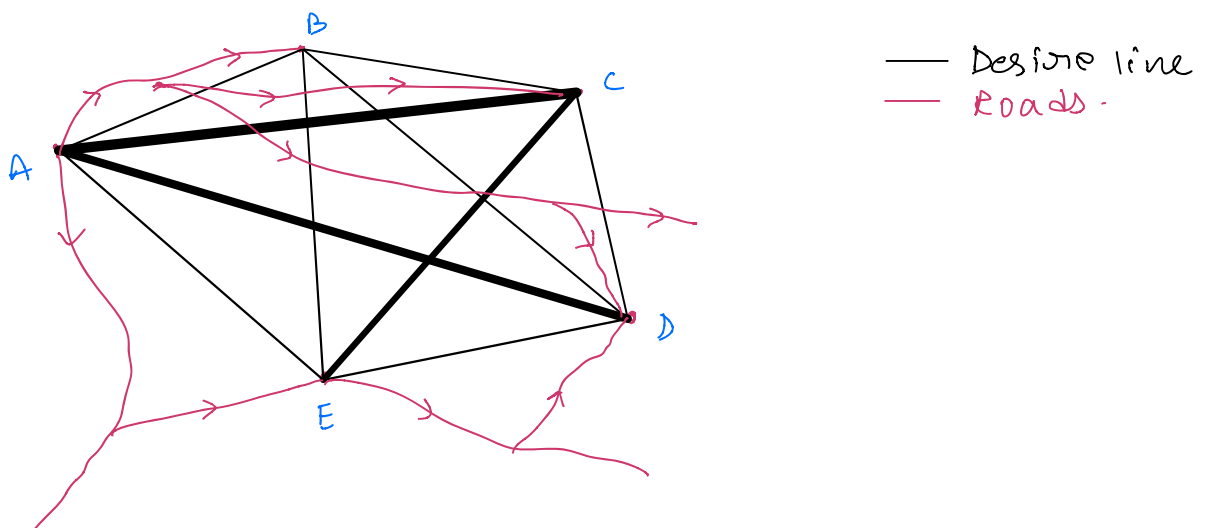
(a) O & D Tables :- are prepared showing no. of trips b/w different zones.

(b) Desire line :- this are plotted which is a graphical representation prepared in O & D studies.

- There are st. lines connecting the origin points with destination.

- The width of such desire line is drawn proportional to number of trips in both directions.

- The desire line density map helps to decide the actual desire of the road user w.r.t the path to be followed b/w two points.



(c) O & D studies data can also be represented in the form of "PIE chart" and "contour lines".

(VI) PARKING STUDIES :-

- All the vehicles don't keep moving along the roadway during the entire 24 hrs of day, they have to stop or park at desired location for different durations, hence parking studies are being carried out to cater the need of parking.
- Parking facilities may be broadly divided into two types.

(a) OFF street parking.

(b) ON street parking. / KERB parking

- (a) OFF STREET parking :- At locations where parking demand is very high and kerb parking cannot be permitted in view of traffic congestion, off street parking is provided, depending upon availability of space.
- In this type of parking vehicles are not allowed to stand in kerb but some distance from it.
 - These are further divided into two:-

(i) Surface parking

(ii) Multi floor parking garage.

- (b) ON STREET parking :- In this type vehicles are parked along the kerb, which may be designed for parking -

- It is convenient for those who could find a suitable space to park the vehicle at desired location.

- Different patterns of kerb parking are -

(i) Parallel parking

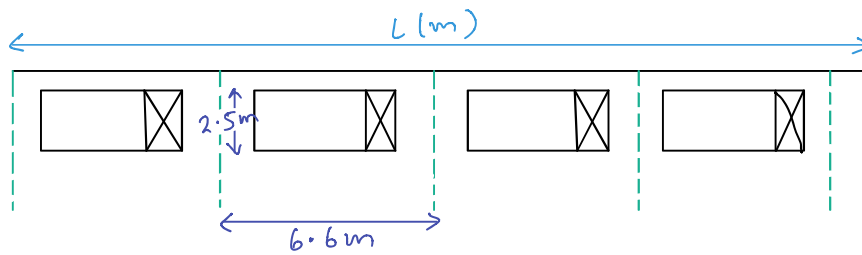
(ii) Angle parking.

Feb 1 (i) Parallel Parking :-

It requires less road width but the number of vehicles that can be parked per unit length of road is least in this case.

- It is preferred when the width of kerb parking space and width of streets are limited.

- In this case parking & unparking are more difficult and time taking.

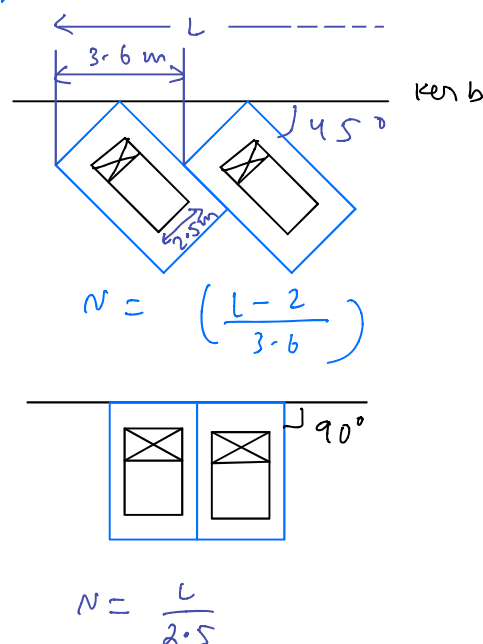
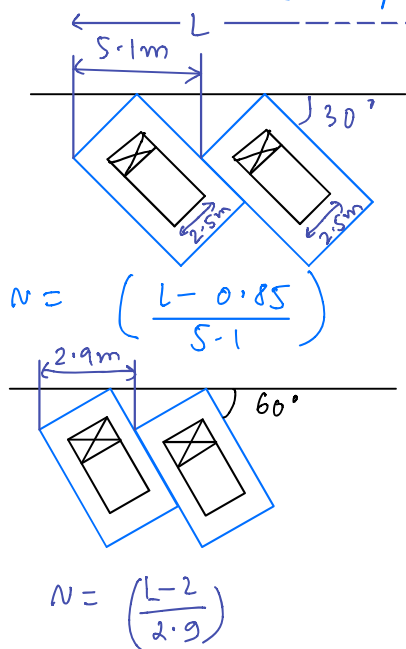


$$N = \frac{L}{6.6}$$

L = length of the kerb.
 N = no. of parking spaces

(ii) Angle parking :-

- In this case vehicles are being parked at an angle ranging from 30° , 45° , 60° and 90° .
- The width of roadway required for parking and unparking width required increases in this case with increase in angle & becomes max at 90° .
- It can accommodate more vehicles per unit length of kerb, with max no of vehicles being parked at 30° .
- This type of parking convenient, but creates more obstruction to the traffic.
- out of various angles for parking, 45° angle parking is considered to be best accounting all the parameters.



(VII) ACCIDENT STUDIES :-

- The problem of accidents is very significant due to complex flow patterns and presence of mixed type of vehicle over the roadway.
- These accidents may involve property damage, personal injuries and may even death also.
- The accident studies are being carried out for following objectives,

(a) To study the cause of accident.

(b) To evaluate existing design and control measures

(c) To evaluate the total loss due to accidents.

(d) To analyse the changes required in existing design.

- The accidents may be caused due to any one or more of the reasons :-

(a) Drivers negligence.

(d) vehicle defect.

(b) Pedestrian.

(e) Road condition.

(c) Passengers.

(f) Road design.

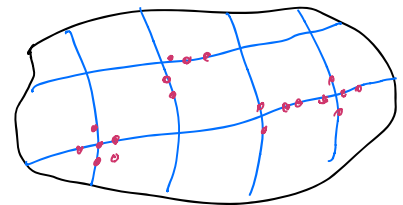
(h) weather (fog, rain)

(g) Traffic condition.

- Accident data can be reported in any of the following forms.

(a) LOCATION FILES :- In this files, location of the accident is being marked which is further used to identify the locations of high accident-rate.

(b) SPOT MAPS :- This maps shows accident by spots pins or symbols on the road map of the locality.



Spot map

- The common symbol used are.

- Type of accident

FATAL

NON-FATAL

Motor vehicle - Pedestrian



Other motor vehicle



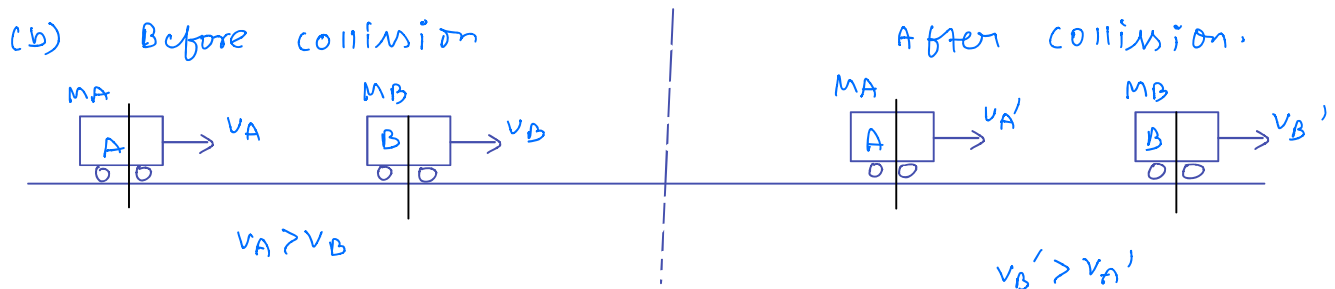
(c) collision diagram:- This diagram depicts the accident location and show the approx path of vehicles and pedestrians involved in accident and also other objects

- Motor-vehicle moving ahead.
- ⇌⇌⇌ Motor-vehicle backing up.
- Pedestrian.
- ▢ → Parked vehicle. → Property damage.
- → Fixed object.
- *→ Rear end collision.
- ~→ out of control.
- *→ Fatal accident.
- *→ Personal injury.

- Analysis of individual traffic accidents can be done on the basis of following assumptions.

(a) when skid marks are present the skid distance are measured to find the actual braking distance and it is assumed 100% skid has occurred due to braking.

- when skid marks are not visible, it may be assumed as free collision without brakes being applied or due to brake failure



$$\begin{aligned} \text{vel. of approach} &= V_A - V_B \\ \text{vel. of separation} &= V_B' - V_A' \end{aligned}$$

As per momentum conservation,

$$\text{Total momentum before impact} = \text{Total momentum after collision.}$$

$$M_A V_A + M_B V_B = M_A V_A' + M_B V_B'$$

NOTE:- Here vel. of sep. can be linked with vel. of approach with the help of coeff. of restitution.

$$e = \frac{\text{vel. of separation}}{\text{vel. of approach}} = \frac{V_B' - V_A'}{V_A - V_B}$$

For elastic collision, $e = 1$

$$v_B' - v_A' = v_A - v_B \Rightarrow \text{vel. of sep} = \text{vel. of approach.}$$

For plastic collision, $e = 0$.

$$v_B' - v_A' = 0 \Rightarrow \text{vel. of sep.} = 0.$$

(c) The impact of vehicle may be direct/oblique at known angle.

(d) Coeff. of friction (f) of pavement surface under the existing condition may either be determined from brake test or is assumed.

- Analysis of initial speed from skid distance is done as follows.

$$\Delta K.E = \text{Work done.}$$

$$\frac{1}{2} m v_2^2 - \frac{1}{2} m v_1^2 = -\text{Friction} \times \text{distance}$$

$$\frac{m}{2} (v_1^2 - v_2^2) = -f m g s$$

$$v_1 = \sqrt{v_2^2 + 2fgs}$$

- If the vehicle stops after skidding the distance "s".

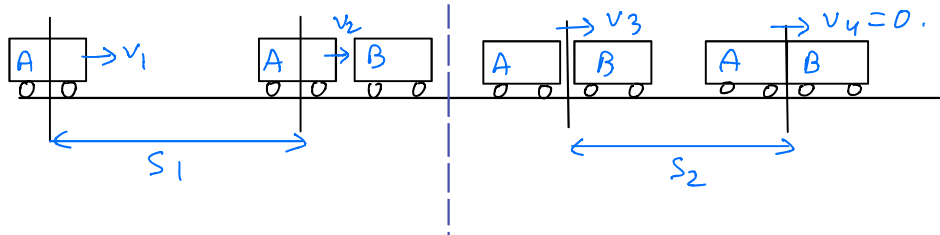
$$\text{hence, } v_2 = 0$$

$$v_1 = \sqrt{2fgs}$$

Feb-3 Accident type-① :- collision of moving vehicle with parked vehicle.

Before collision

After collision.



Properties	veh A	veh B
(i) Initial speed	v_1	0
(ii) Speed just bef. collision	v_2	0
(iii) " Just after collision	v_3	v_3
(iv) Skid before collision	s_1	0
(v) Skid after collision	s_2	s_2
(vi) Mass	m_A	m_B

Step ① :- Before collision :-

$$\Delta KE = WD$$

$$\left(\frac{1}{2} m v_2^2 - \frac{1}{2} m v_1^2 \right)_A = -f m g s_1$$

$$v_1^2 = v_2^2 + 2f g s_1 \quad - (1)$$

Step ② :- At collision :-

Momentum before = Momentum after.
collision collision.

$$m_A v_2 + m_B \cdot 0 = m_A v_3 + m_B v_3$$

$$v_2 = \frac{(m_A + m_B)}{m_A} v_3 \quad - (2)$$

From ① and ③

$$v_1^2 = \left(\frac{m_A + m_B}{m_A} \right)^2 v_3^2 + 2f g s_1 \quad - (3)$$

Step ③ :- After collision :-

$$\Delta KE = WD.$$

$$\frac{1}{2} (m_A + m_B) \cdot 0^2 - \frac{1}{2} (m_A + m_B) v_3^2 = -(m_A + m_B) g f s_2.$$

$$v_3^2 = 2g f s_2 \quad - (4)$$

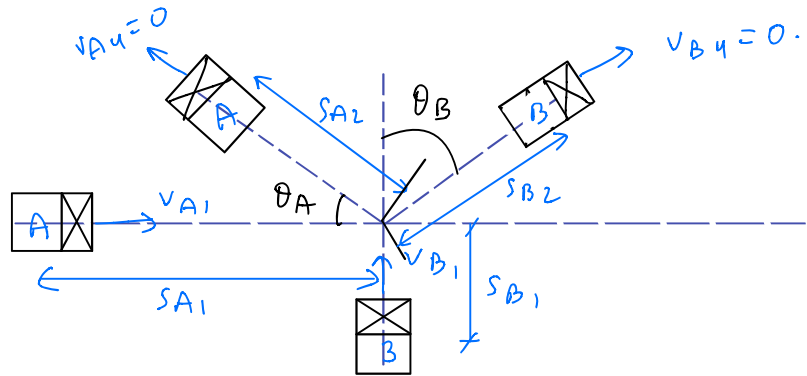
From eqn (3) + (4)

$$v_1^2 = \left(\frac{m_A + m_B}{m_A} \right)^2 (2g f s_2) + 2g f s_1$$

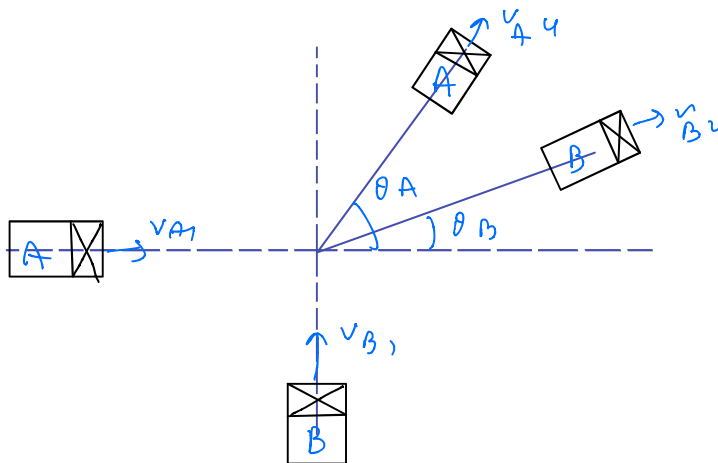
$$v_1 = \sqrt{\left[\left(\frac{m_A + m_B}{m_A} \right)^2 s_2 + s_1 \right] 2g f}$$

ACCIDENT TYPE-(ii) :- Two vehicles approaching right angle.

(case a)

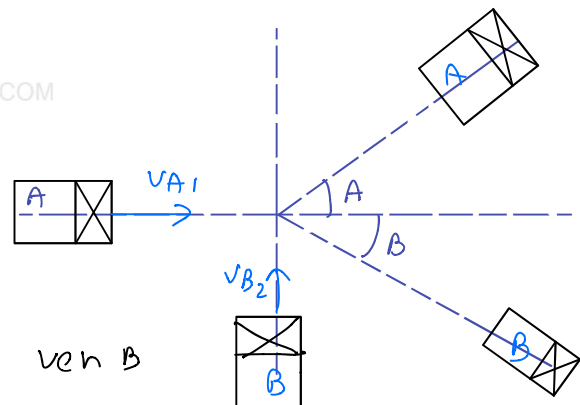


(case b)



(case c)

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Properties

veh A

veh B

Initial speed

v_{A1}

v_{B1}

Speed just before collision.

v_{A2}

v_{B2}

speed just after collision.

v_{A3}

v_{B3}

skid just before collision

s_{A1}

s_{B1}

skid just after collision

s_{A2}

s_{B2}

mass of vehicle

m_A

m_B

STEP ① :- Before collision :-

$$\Delta KE = W_D$$

$$\frac{1}{2} m v_2^2 - \frac{1}{2} m v_1^2 = -m g f s$$

$$v^2 = v_2^2 + 2 g f s$$

for veh A,

$$v_{A1}^2 = v_{A2}^2 + 2 g f s_A,$$

for veh B, $v_{B1}^2 = v_{B2}^2 + 2 g f s_B,$

step ② :- During collision :-

Momentum just before collision = mom. just after coll. in a particular dir.

case (a) :-

x-axis :- $m_A v_{A2} + 0 = -m_A v_{A3} \cos \theta_A + m_B v_{B3} \sin \theta_B$

$$v_{A2} = \frac{m_B}{m_A} v_{B3} \sin \theta_B - v_{A3} \cos \theta_A.$$

y-axis :- $m_B v_{B2} + 0 = m_A v_{A3} \sin \theta_A + m_B v_{B3} \cos \theta_B.$

$$v_{B2} = \frac{m_A}{m_B} v_{A3} \sin \theta_A + v_{B3} \cos \theta_B$$

case - (b) :-

$$m_A v_{A2} + 0 = m_A v_{A3} \cos \theta_A + m_B v_{B3} \cos \theta_B$$

$$v_{A2} = v_{A3} \cos \theta_A + \frac{m_B}{m_A} v_{B3} \cos \theta_B$$

$$m_B v_{B2} + 0 = m_A v_{A3} \sin \theta_A + m_B v_{B3} \sin \theta_B$$

$$v_{B2} = \frac{m_A}{m_B} v_{A3} \sin \theta_A + v_{B3} \sin \theta_B.$$

case (c) :- $m_A v_{A2} + 0 = m_A v_{A3} \cos \theta_A + m_B v_{B3} \cos \theta_B.$

$$v_{A2} = \checkmark$$

$$m_B v_{B2} + 0 = m_A v_{A3} \sin \theta_A - m_B v_{B3} \sin \theta_B$$

$$v_{B3} = \checkmark$$

step ③ :- After collision.

$$\Delta KE = W_D$$

$$\frac{1}{2} m 0^2 - \frac{1}{2} m v_3^2 = -mg f S_2$$

$$v_3 = \sqrt{2g f S_2}$$

$$\text{for veh A :- } v_{A3} = \sqrt{2g f S_{A2}}$$

$$\text{for veh B :- } v_{B3} = \sqrt{2g f S_{A3}}$$

- 8) A vehicle of weight 2 tonnes skids through a distance equal to 40m before colliding with another parked vehicle of wt. 1 tonne. After collision both the vehicle skid through a distance equal to 12m before stopping. Compute the initial speed of the moving vehicle. Assume $\mu = 0.5$.

Sol =

$$e = 0 \quad m_A = 2 \text{ tonne}$$

$$m_B = 1 \text{ tonne}$$

$$\mu = 0.5$$

$$v_{A1} = ?$$

$$AKE = WD$$

$$\frac{1}{2} m_A v_{A2}^2 - \frac{1}{2} m_A v_{A1}^2 = -f m g S_1$$

$$v_{A1} = \sqrt{v_{A2}^2 + 2 f g S_1} \quad \text{--- (i)}$$

Momentum eqn. during collision

$$m_A v_{A2} + m_B (0) = (m_A + m_B) v_{A3}$$

$$v_{A2} = \frac{m_A + m_B}{m_A} v_{A3}$$

Energy eqn after collision

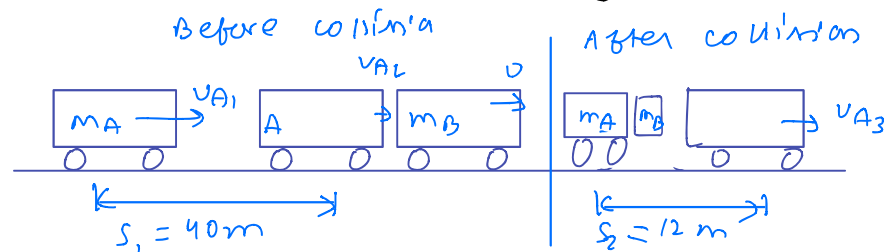
$$\frac{1}{2} m 0^2 - \frac{1}{2} m v_{A3}^2 = -f g m S_2$$

$$v_{A3} = \sqrt{2 f g S_2} = \sqrt{2 \times 0.5 \times 9.81 \times 12}$$

$$= 10.84 \text{ m/s}$$

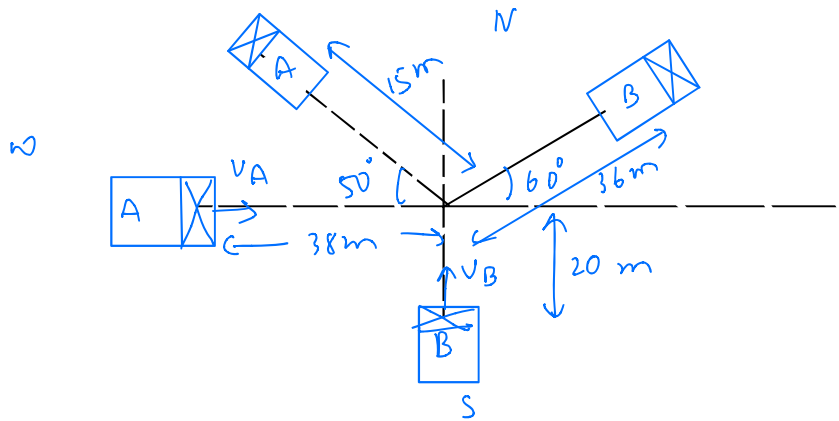
$$v_{A2} = 16.2 \text{ m/s}$$

$$v_{A1} = 25.58 \text{ m/s.}$$



- Q) Two veh. A & B approaching at right angles. A from west and B from south collide with each other. After collision veh. A skids in a direction 50° north of west and veh. B 60° east of north. The initial skid distance of veh. A and B are 38m and 20m respectively before collision. The skid distance after collision are 15m & 36m respectively. If the weight of veh. A & B are 4 & 6 tonnes, compute the original speed of vehicle. $f = 0.55$

Sol =



$$W_A = 4 \text{ tonne.}$$

$$W_B = 6 \text{ tonne.}$$

$$F = f = 0.55$$

Before collision :-

$$\frac{1}{2} m_A v_{A2}^2 - \frac{1}{2} m_A v_{A1}^2 = -f g m_A (38)$$

$$v_{A1}^2 = v_{A2}^2 + 1640.2$$

During collision :-

$$x\text{-axis : } m_A v_{A2} = -m_A v_{A3} \cos 50^\circ + m_B v_{B3} \cos 60^\circ$$

$$v_{A2} = \frac{m_B}{m_A} v_{B3} \cos 60^\circ - v_{A3} \cos 50^\circ$$

$$v_{A2} = \frac{3}{4} v_{B3} - v_{A3} (0.642)$$

$$y\text{-axis :- } m_B v_{B2} =$$

21.98.

- The various measures that can be taken to reduce the accidents which are categorised into following.

(a) Engineering

(b) Enforcement

(c) Education.

(i) Designing Road

(i) Speed limit

(i) Awareness

(ii) Maintenance of veh.

(ii) Traffic control

(ii) Education of

(iii) Before and after studies

(iii) Medical checks

(iii) Road user.

(iv) Road lighting.

(iv) For commercial vehicles.

(iii) Safety drive.

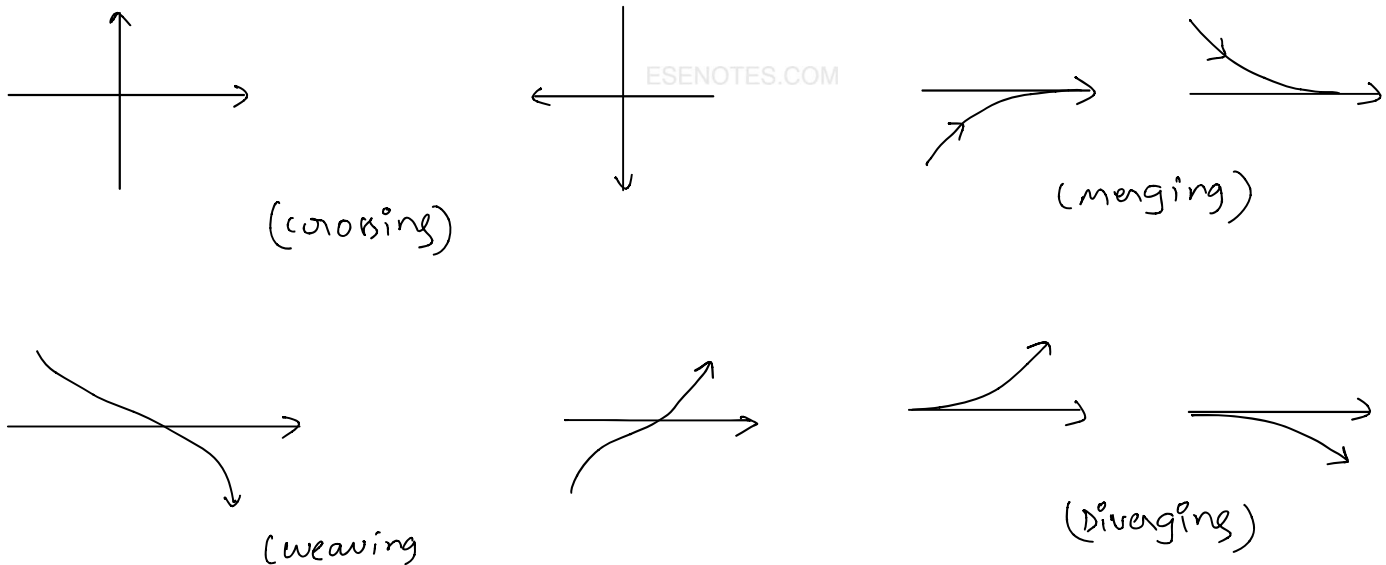
TRAFFIC CONTROL DEVICES & REGULATION

(i) INTERSECTION :-

- An intersection is where two or more roads joins or cross.
- At the intersection, there are through, turning and crossing traffic movements.
- This movements of traffic are handled depending upon the type of intersection.
- Intersections are broadly classified into two.

(a) Intersection at grade.

- When all the roads meet at about the same level, allowing the traffic movements like crossing, weaving, merging & diverging are called intersection at grade.

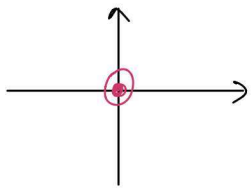


- The basic requirements of intersection at grade are
 - (a) At the intersection the area of conflict should be minimum
 - (b) The relative speed of vehicle and angle of approach should be minimum
 - (c) Adequate visibility should be available for all the vehicles.
 - (d) Good lighting at night is desired.

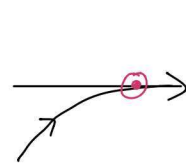
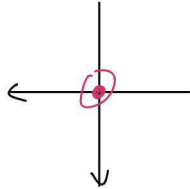
(e) proper sign should be installed

(a) The point where the possible path of two vehicles intersect is called "conflict point"

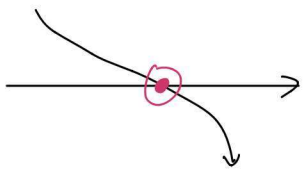
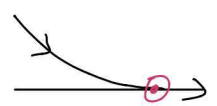
The area containing all possible conflict points is termed as "conflict area."



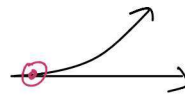
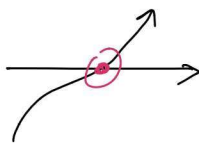
(crossing)



(merging)



(weaving)



(diverging)

- If the relative speed and $\angle$ of approach of two vehicles is more it increases the possibility of collision b/w the vehicle and these conflict point are termed as major conflict points.

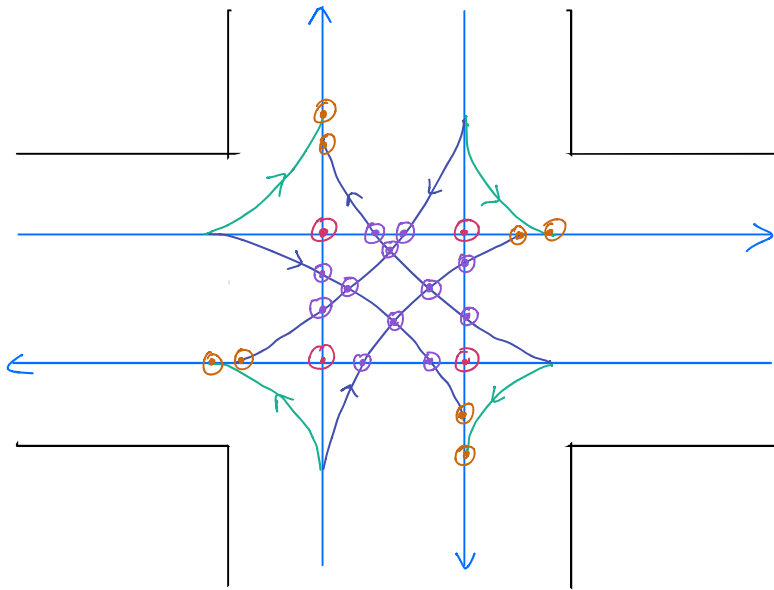
eg:- crossing and weaving conflict.

- If rel. vel & $\angle$ of approach of two vehicles is less, possibility of collision decreases and these are termed as minor conflict points.

eg:- merging & diverging conflict.

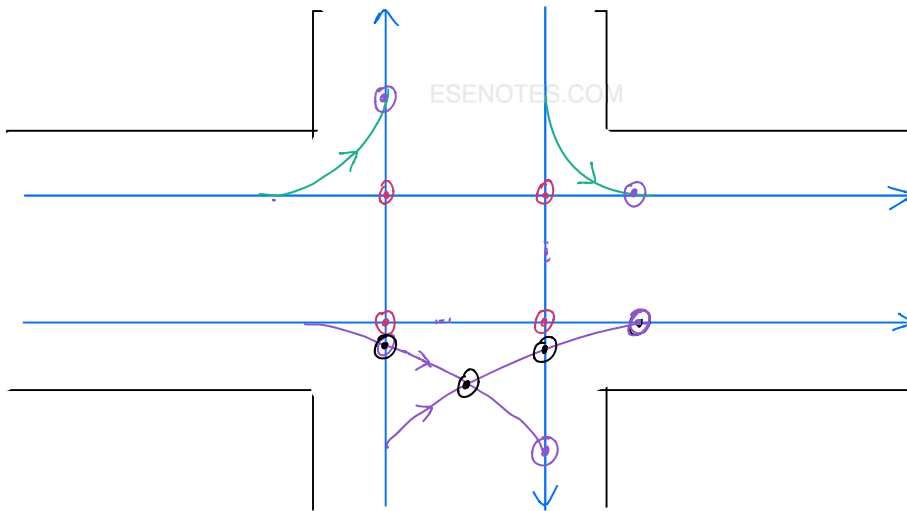
- ⊛ - IRC do not consider diverging conflict.

case 1:- conflict on cross roads with two way traffic on both roads.



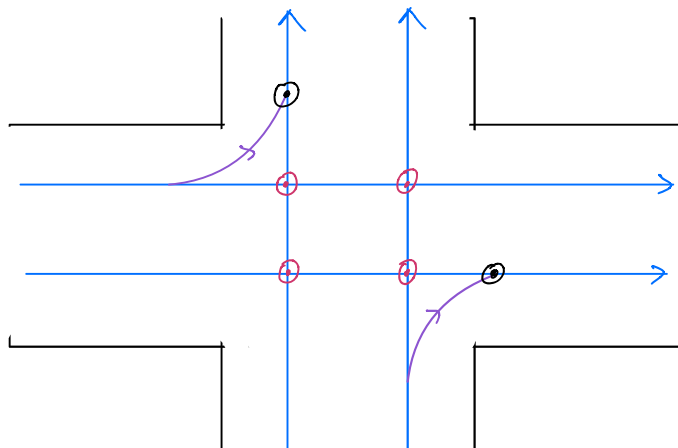
$$\begin{array}{l} \text{crossing (c)} = 4 \\ \text{weaving (w)} = 12 \\ \text{merging (m)} = 8 \\ \hline 24 \end{array}$$

case-(II) conflict on cross roads with one-way regulation on one road.



$$\begin{array}{l} c = 4 \\ w = 3 \\ m = 4 \\ \hline 11 \end{array}$$

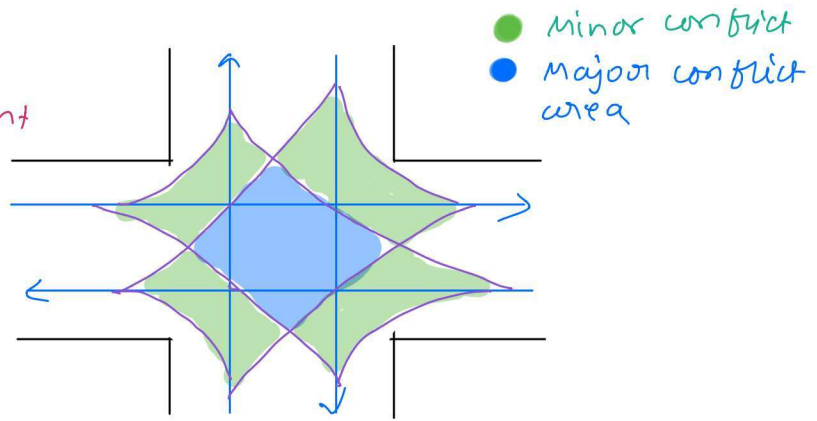
case (iii) conflicts on cross roads with one way regulation on roads.



$$c = 4$$

conflict Area

NOTE:- Number of conflict point for diff road comb'



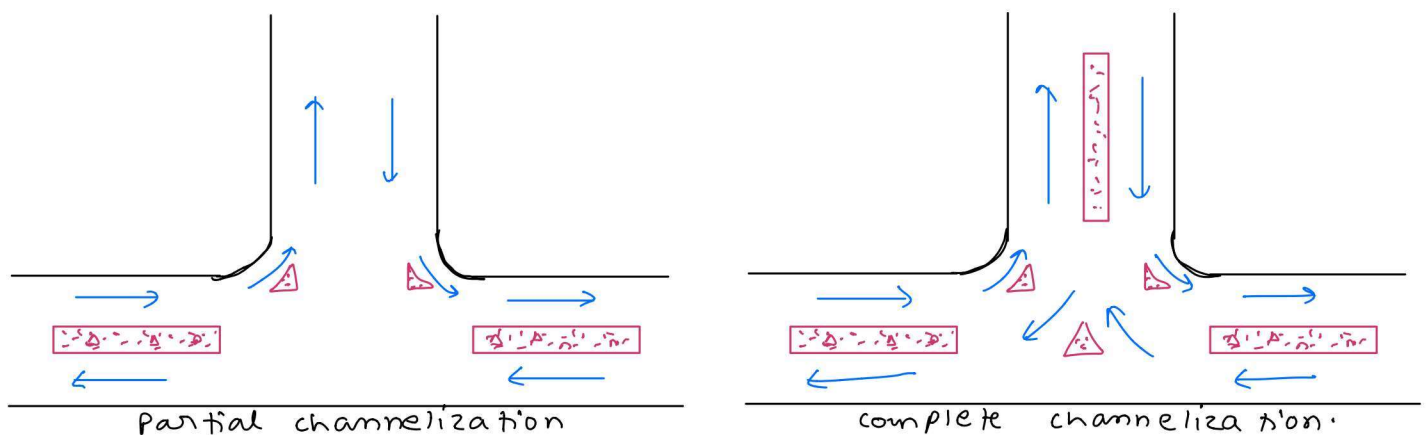
Number of lanes		Number of potential conflicts.		
Road 1	Road 2	Both road two way	1: One way 2: Two way	Both road one way.
2	2	24 (Ma: 16 Mi: 8)	11 (Ma = 7 Mi = 4)	6 (Ma = 4 Mi = 2)
2	4	32	17	10
4	4	44	25	18

- At grade intersections are further of following types.

(a) UNCHANNELISED INTERSECTION:-

- In this entire intersection is paved and there is no restriction to the vehicles to use any part of this intersection area.
- It is very easy to construct but has most complex traffic operations over it resulting in large conflict area.
- It is suitable to be provided where traffic volume is low.

(b) CHANNELISED INTERSECTION



- It is achieved by introducing islands into the intersecting or intersection area in order to channelize the traffic flow into different streams.
- It reduces the total conflict area available.
- It does not reduce conflict points.
- It is comparatively diff to construct.
- channelizing islands provided in this case serves the following purpose.
 - (a) Reduces relative speed.
 - (b) Reduces relative angle.
 - (c) Reduces conflict area

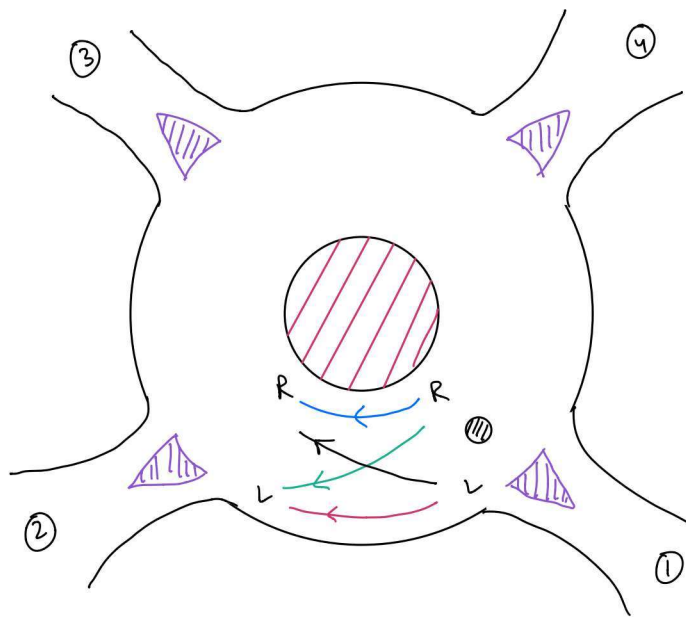
Feb-8

(c) ROTARY INTERSECTION :-

It is an enlarged road intersection where all the converging vehicles are forced to move around a large central island in one direction before they can weave out of traffic flow into their respective direction radiating from central island.

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- The main objective of providing a rotary is to eliminate the need of stopping even for crossing streams of vehicle and to reduce the area of conflicts.
- By providing rotary crossing conflict is eliminated and converted into weaving conflict which consists of following -
 - (movement)
 - (i) merging maneuver from the left and diverging out to the right. (Black line)
 - (ii) merging from right and a diverging out to the left (Green line).



R-R	$\frac{V_{43}}{V_{12}}$	"d"
L-L	$\frac{V_{12}}{V_{43}}$	"a"
R-L	$\frac{V_{4-2}}{V_{3-2}}$	"c"
L-R	$\frac{V_{1-3}}{V_{1-4}}$	"b"

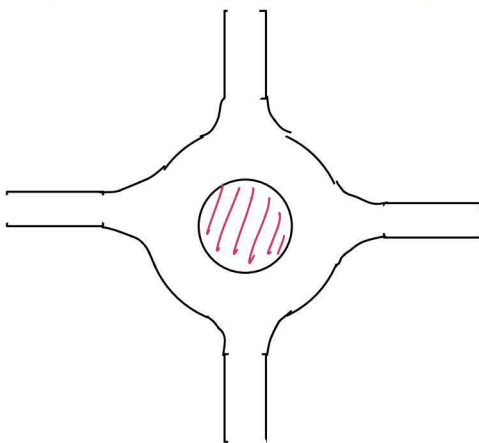
- Design factors for rotary intersection :-

(a) DESIGN SPEED :- vehicles approaching at grade intersection have to considerably slow down their speed as compared to design speed of that particular road, however there is no need for vehicles to stop at rotary.

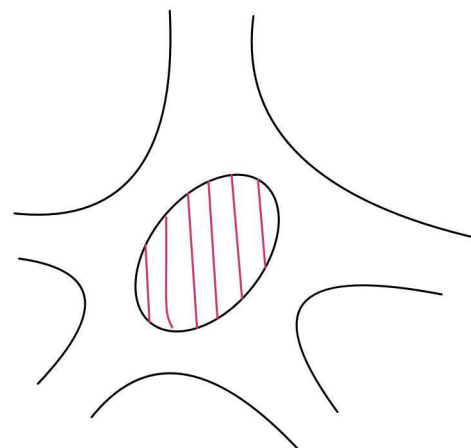
- For rural areas, design speed = 40 kmph
 urban areas " " = 30 kmph.

NOTE:- Since in India we follow "keep to left" notations flow of vehicle around central island is in clockwise direction.

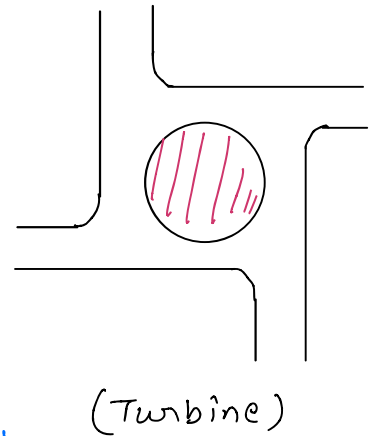
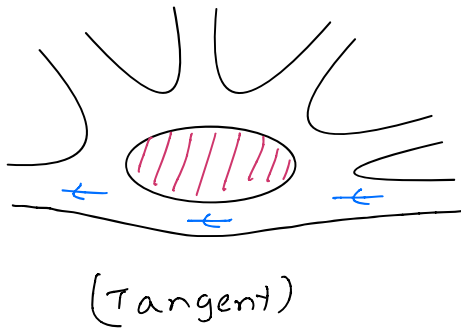
(b) SHAPE OF ISLAND :-



(Circular)



(Elliptical)



- The shape of central island depends on the number and layout of intersecting roads.
- When two equally imp. roads cross at roughly right angles i.e. all the four radiating roads are placed symmetrically, circular shape is suitable.
- The island may be elongated to accommodate in layout where more than four intersecting roads are there termed as elliptical shape.
- "TANGENT SHAPE" is not desirable as there is tendency of traffic in this direction to move much faster than specified speed.
- "TURBINE SHAPE" forces reduction in speed of vehicle entering the rotary, but creates the problem at due to headlight glare.

(C) RADIUS OF CENTRAL ISLAND & ROTARY

As in case of normal horizontal curve, centrifugal force is resisted by friction and super-elevation.

Similarly in case of rotary roadway it is resisted only by friction as super-elevation cannot be provided in this case

$$e + f = \frac{v^2}{127 R}$$

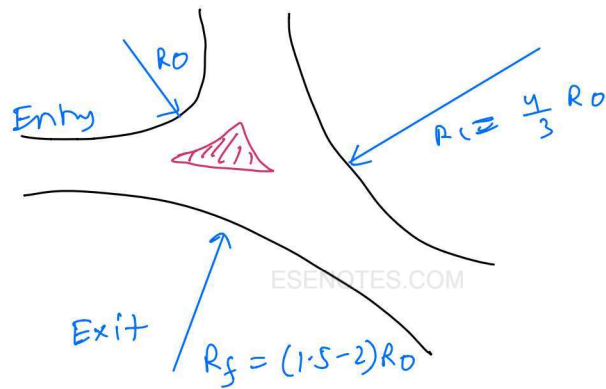
$$f = \frac{v^2}{127 R}$$

$$R = \frac{v^2}{127 f} \rightarrow \text{Kmph}$$

Hence, radius of entry curve is as follows.

Type of road	Design speed v (kmph)	Radius (m)
Rural	40	20-35
Urban	30	15-25

- IRC recommends min. radius of central island to be 1.33 times of radius of entry curve.
- Vehicles leaving the rotary would accelerate to the speed of radiating road and hence the exit curve should be of larger radius than entry curve.
- IRC recommends radius of entry curve to be (1.5-2) times the radius of entry curve.

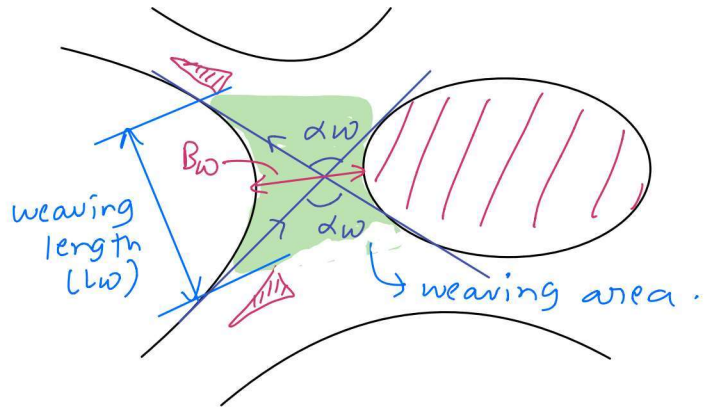


(d) Weaving Area:- Since by providing rotary crossing conflict is converted to weaving movement, which takes place in zone termed as weaving area. Hence its design becomes significant.

- For smooth traffic flow, the weaving angle should be small but not less than 15° as it would increase dia of central island.
- The weaving length should be atleast four times the width of weaving section.
- The recommended value of weaving length is (45-90)m for design speed of 40 kmph and (20-60)m for 30 kmph.

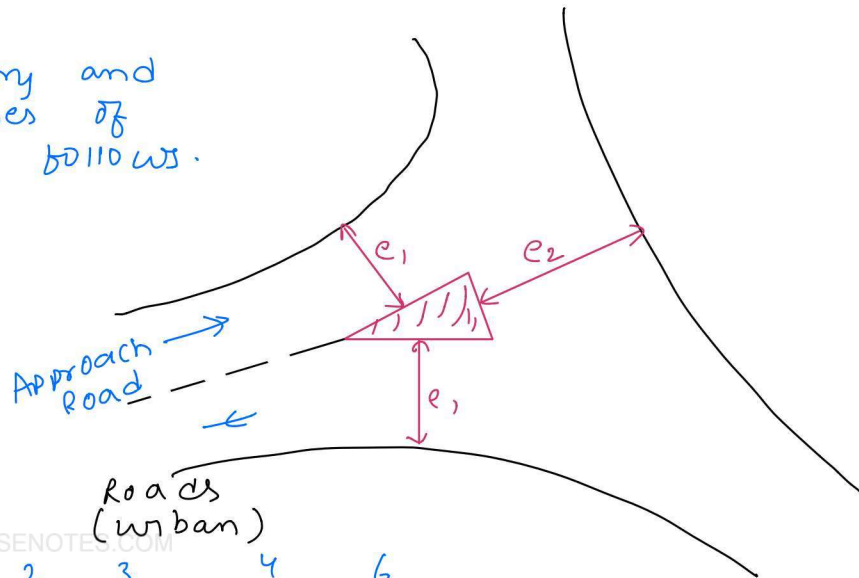
α_w = maximum weaving angle

B_w = weaving width



(e) width of carriageway :-

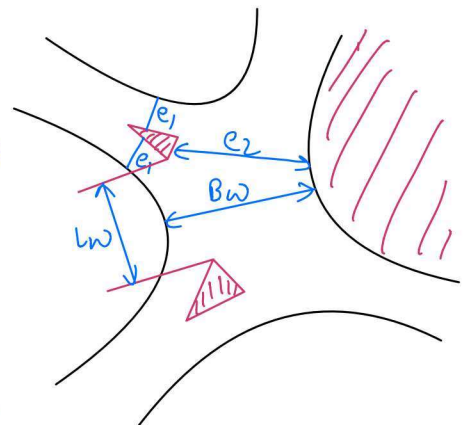
- As per IRC width at entry and exit for different lanes of approach road are as follows.



no. of lanes	Road (Rural)				Roads (Urban)			
	2	3	4	6	2	3	4	6
width of carriageway at entry and exit (m) (e_1)	6.5	7	8	13	7	7.5	10	15

Feb-9 (f) width of Rotary roadway :-

- The width of non-weaving section (e_2) of the rotary should be equal to widest single entry to the rotary & should generally be less than the width of weaving section. ($e_2 < B_w$)
- The width of weaving section (B_w) of the rotary should be one traffic lane wider than the mean width of the entry and non weaving section.



$$\therefore B_w = \left[\frac{e_1 + e_2}{2} + 3.5 \right] = e + 3.5$$

CAPACITY OF ROTARY :-

The practical capacity of rotary is dependent upon min capacity of the individual weaving section and is given by following expression.

$$Q_p = \frac{280 B_w \left(1 + \frac{e}{B_w}\right) \left(1 - \frac{p}{3}\right)}{\left(1 + \frac{B_w}{L_w}\right)}$$

Q_p = practical capacity of weaving section of rotary (PCU/hr)

B_w = width of weaving section. (6-18)m

e = Avg width of entry (e_1) & width of non-weaving section e_2

$$e = \frac{e_1 + e_2}{2}$$

p = proportion of weaving traffic

NOTE!- Here $\left(\frac{e}{B_w} = 0.4 - 1\right)$

$$p = \frac{b+c}{a+b+c+d} \quad p = (0.4 - 1.0)$$

Not participating in weaving.
 { a = left turning traffic from extreme left lane.
 d = Right turning traffic from extreme right lane.
 Participate in weaving
 { b = weaving traffic turning towards right while entering the rotary.
 c = weaving traffic turning left while leaving the rotary.

- As per IRC, the following PCU values for diff. types of vehicles are as follows:-

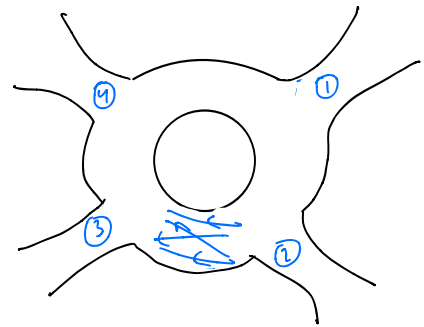
	PCU
(i) Car, LCV, Three wheeler	= 1
(ii) Buses, MCV,	= 2.8
(iii) Motor cycles, scooter	= 0.75
(iv) Pedal cycle	= 0.5
(v) Animal drawn vehicle	= 4-6.

8) Four legs in a rotary intersection are designated as 1, 2, 3, 4. The traffic volume (V_{ij}) in terms of (pcu/hr) are given as follows.

- compute the proportion of weaving traffic to total traffic in the weaving section b/w legs 2 & 3.
- Also indicate the use of this value.

V_{12}	150	V_{21}	310	V_{31}	150	V_{41}	30
V_{13}	450	V_{23}	200	V_{32}	570	V_{42}	1080
V_{14}	412	V_{24}	1090	V_{34}	240	V_{43}	600

Sol weaving traffic : $V_{13} = 450$
 $V_{24} = 1090$
 $V_{21} = 310$
 $V_{43} = 600$
 Non-weaving traffic, $V_{14} = 412$
 $V_{23} = 200$



$$\text{proportion} = \frac{450 + 1090 + 310 + 600}{412 + 1090 + 310 + 200 + 412 + 600} = 0.80$$

9) A round about is provided with an avg entry width of 8.4m, width of weaving section as 14m and length of weaving section b/w channelling island as 35m. The crossing traffic and total traffic on weaving section are 1000 and 2000 PCU/hr.

Find the capacity of round about in (PCU/hr)

Sol $e = 8.4\text{m}$ $L_w = 35\text{m}$
 $B_w = 14\text{m}$
 $p = \frac{\text{crossing}}{\text{Total}} = \frac{1000}{2000} = 0.5$

$$Q_p = \frac{280 B_w \left(1 + \frac{e}{B_w}\right) \left(1 - \frac{p}{3}\right)}{1 + \frac{B_w}{L_w}}$$

$$= \frac{280 \times 14 \times \left(1 + \frac{8.4}{14}\right) \left(1 - \frac{0.5}{3}\right)}{1 + \frac{14}{35}} = 3733.3 \text{ PCU/hr}$$

(g) channelizing Island :-

These are provided at the entry & exit of the rotary for the following purpose.

- (i) To reduce the conflict area
- (ii) To force the vehicles to reduce their speed
- (iii) To provide space for erecting traffic signs.
- (iv) To improve the aesthetic.

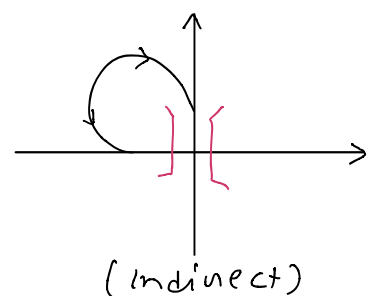
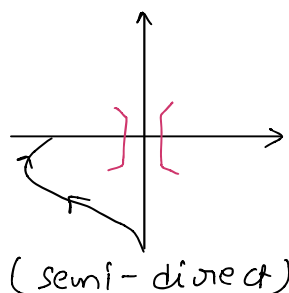
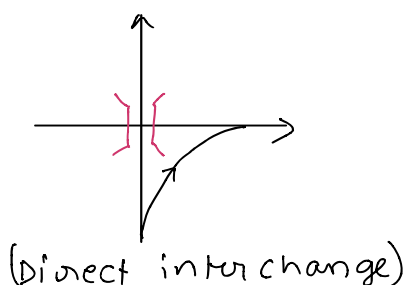
- The islands are provided with kerbs of 150-200mm high.

(h) SIGNALISED INTERSECTION :-

- Intersection at grade that are controlled by traffic signals are called signalised intersection.
- The automatic traffic control signals are operated to alternately stop and let go the traffic entering the intersection from the approach legs, thus preventing the conflicts.

(b) GRADE SEPARATED INTERSECTION

- It is the highest form of intersection available
- Grade separation is achieved by means of vertical level separation of intersecting roads by means of providing
 - (i) an over bridge (fly over)
 - (ii) by under pass
- Transfers of the route at grade separation or turning facilities are provided by "INTERCHANGE"
- Interchange may be of following types :-



Flyover :-

- No drainage problem
- lateral expansion is easier
- vertical profile will be totally disturbed.
- construction cost is less.
- limit heavy vehicle

Underpass :-

- drainage prob
- expansion is diff.
- not aesthetic.
- no such limit.
- cost is less than overpass.

- Interchanges can also be classified on the basis of shape as follows -

(a) DIAMOND

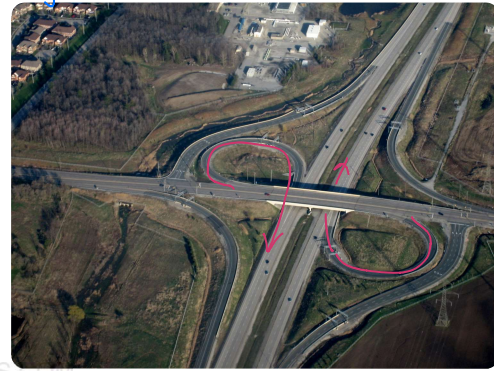
(c) Partial clover leaf.

(b) Rotary

(d) Full clover leaf.



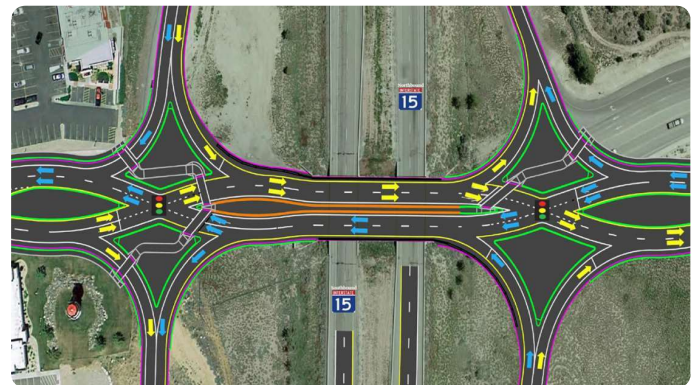
Full clover leaf



Partial clover leaf



Rotary interchange



DIAMOND INTERCHANGE

Feb 12

#(E) TRAFFIC CONTROL DEVICES

- The devices placed in order to control the traffic flow is termed as traffic control devices.

Eg:- Traffic signals, signs, pavement marking etc.

- These devices are classified as follows:-

(i) Regulatory devices:-

- These give the road user notice of traffic laws or regulations that apply to given
- Disregard of such devices is punishable as an violation. Eg. STOP, no turning, no parking etc.

(ii) WARNING DEVICES:-

- These call attention of the road user to the condition on the roadway, that are potentially hazardous to traffic operation.
Eg narrow road ahead, slippery road etc.

(iii) GUIDING / INFORMATORY:-

- These provide directions & information to the road user regarding distances, destinations,

(I) TRAFFIC SIGNAL:-

- These are control devices which could alternately direct the traffic to stop & proceed at intersection using red and green traffic light signals.

- The main required of traffic signals are:-

- (a) It should draw the attention of road user.
- (b) " " provide meaning & time to respond.
- (c) " " have minimum waste of time.

- Traffic signals are of following types:-

(a) TRAFFIC CONTROL SIGNALS:-

- These signals are provided to control the traffic at the intersection and have three coloured light, facing each direction of traffic flow

- These are further classified into three.

(a) Fixed time / Pre timed :- These are said to repeat regularly a cycle of red, amber and green lights. The timing of each phase is pre-determined.

- The main drawback of these signals is that some times traffic flow on one road may be almost nil and traffic on cross-road may be quite heavy, but time available in both cases is same.

(b) Semi-Actuated :- These are signals whose timing is affected when vehicles are detected by video, sensors on some but not all the approaches.

(c) Fully Actuated signals :- These are the signals timings of which is completely influenced by the traffic volume when detected on all the approaches.

(b) PEDESTRIAN SIGNALS :- These signals are provided to control the movement of pedestrian along the approaches to intersection.

(c) SPECIAL TRAFFIC SIGNALS :- These signals are provided to aware the road user about the condition prevailed ahead on road way
Eg. BEACON.

- Types of coordination of traffic signal system consist of the following :-

(i) Simultaneous system :- In this system all signal along a given road always show same indication.

(ii) Alternate system :- In this system alternate signals or group of signals show opposite indication in a route at same time.

(iii) Simple progressive system :- It is the type of system which permits as nearly as possible a continuous operation of group of vehicles along the main road at a reasonable speed.

(iv) flexible progressive system :- In this system it is possible to automatically vary the length of cycle divisions and the time scheduled at signalised intersection.

- Most efficient amongst all system.

- Traffic signal should not be installed unless one or more of the following signal warrant are met

(a) Minimum vehicular volume warrant.

- The avg traffic volume for 8 hr of the day on both approaches should be atleast 650 veh/hr on major streets with single and 800 vehicles on streets with two or more lanes.

(b) Interruption of continuous traffic flow on major roads.

- when the traffic flow on major street is 1000-1200 veh/hr and there is undue delay on minor road with traffic of (100-150) veh/hr in one direction only during any 8 hrs of the day. traffic signals are required.

(c) Min. Pedestrian volume warrant.

- when 150 or more pedestrian per hour cross a major road with over 600 vehicles per hour on both approaches then traffic signals are required.

(d) Accident experience warrant.

- If the accident record shows that other measures are not useful to decrease the accident frequency in last 12 month, signal is provided.

NOTE :- when no single warrant is satisfied but indicates that two or more warrants of a, b, c, are 80% or more satisfied then traffic signals are provided.

ANALYSIS OF TRAFFIC AT SIGNALISED INTERSECTION

(i) CYCLE :- A signal cycle is one complete rotation of all the indications provided. (RED, AMBER, GREEN).

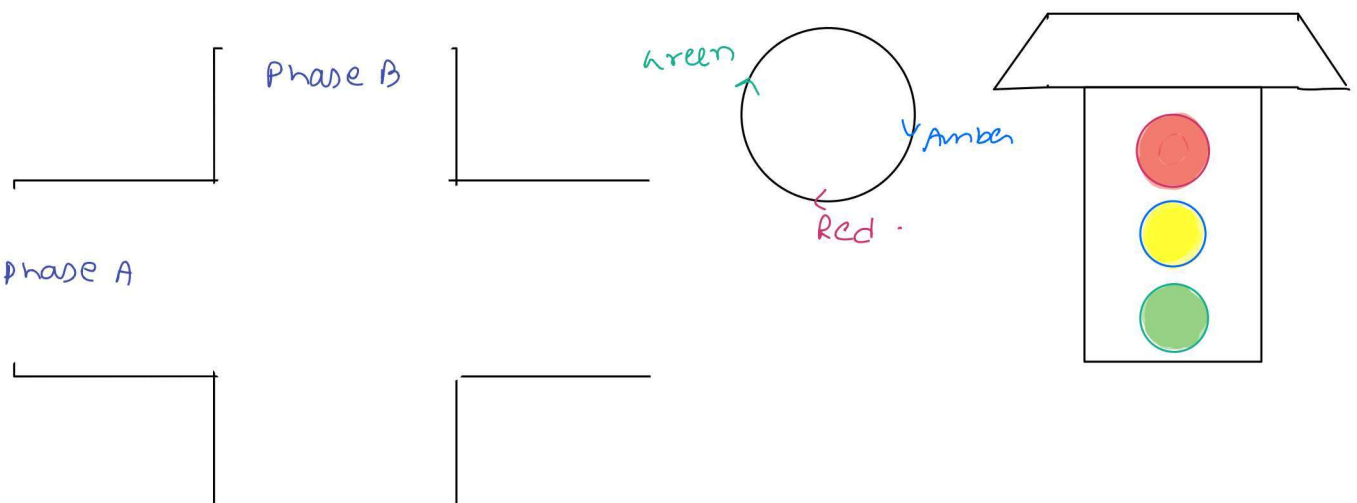
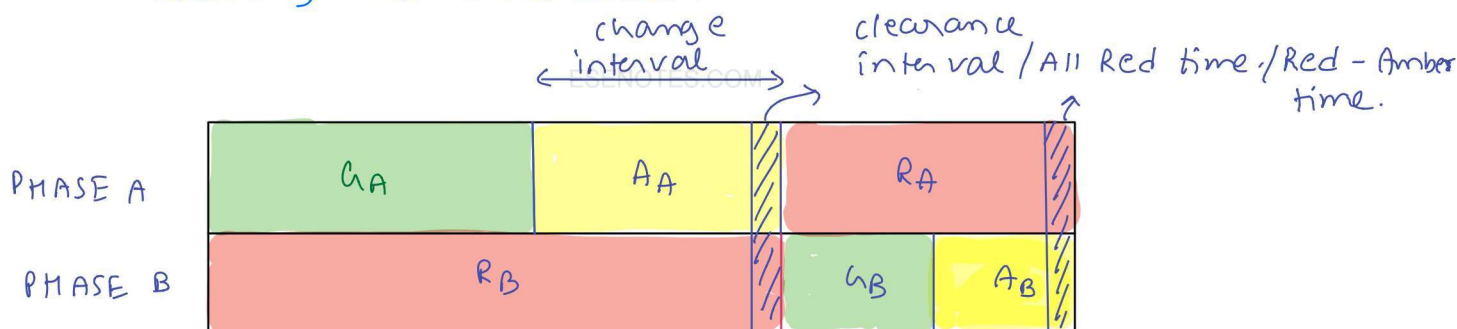
(ii) CYCLE LENGTH :- It is the time in seconds that it takes a signal to complete one full cycle of indications i.e. time interval between starting of green for one approach till the next time the green appears.

(iii) INTERVAL :- It represents change from one stage to other. These are of two types.

(a) CHANGE INTERVAL :- It represents yellow/amber time.

- It indicates the interval b/w green and red signal.

(b) CLEARANCE INTERVAL :- It represents all red time. It is included after each yellow interval indicating a period during which all signal phase show red and is used for clearing the intersection.



$$C = G_A + A_A + R_A \text{ --- (i)} \quad G_A + A_A = R_B \text{ --- (iii)}$$

or

$$C = G_B + A_B + R_B \text{ --- (ii)} \quad G_B + A_B = R_A$$

Hence, $C = R_B + R_A \rightarrow \text{All red}$

$$\begin{aligned} \therefore C &= (G_A + A_A) + (G_B + A_B) \\ &= \underbrace{(G_A + G_B)}_{\text{All green}} + \underbrace{(A_A + A_B)}_{\text{All Amber}} \end{aligned}$$

- Thus cycle time is All RED time
Sum of All GREEN and All AMBER.

NOTE:- clearance time/interval is provided after yellow interval and is optional. Hence if intersection is small then there is no need of clearance interval.

(iv) GREEN TIME/INTERVAL (G_i):- It is provided to allow traffic flow through the intersection. It is designed on the basis of traffic volume of given road.

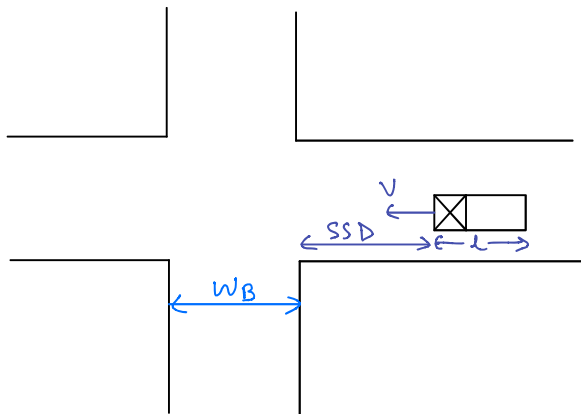
- It is actual time for which green signal is turned on.

(v) RED TIME/INTERVAL:- It is provided to stop the traffic flow on the intersection.

- It is designed on the basis of cross road traffic volume

- It is the actual duration the red light is turned on.

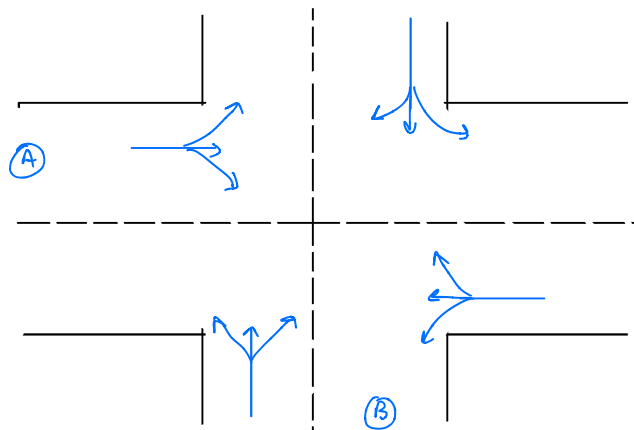
(vi) AMBER TIME:- It is the clearance time provided after the green time, just for clearance of traffic, so that traffic on the cross-road can be started.



$$\text{Amber time} = \frac{W_B + SSD + L}{V}$$

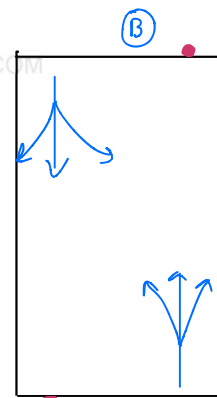
Feb 13 (vii) PHASE:- The phase is defined as the sum of displayed green, yellow, red times for a movement or combination of movement that receive the RIGHT of way simultaneously during the cycle

- The sum of phase length is the cycle length.
- In the traffic movement at intersection following type of phases may be observed.

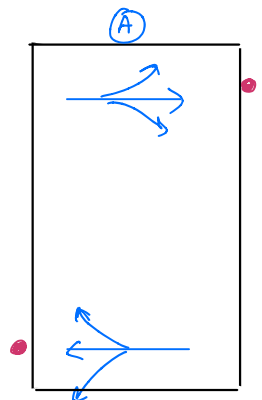


(a) Two phase operation :-

- This system is adopted if through traffic is significant compared to turning movements.
- If turning movements are significant then 4-phase system is provided.



Phase-(I)

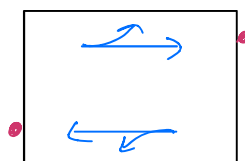


Phase-(II)

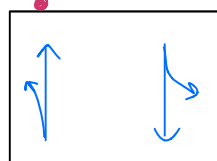
- which can be given through different phase

(b) 4-phase operation :-

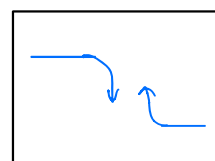
case (I) :-



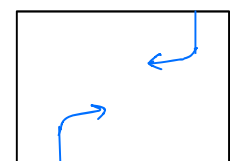
Phase-(I)



Phase-(II)



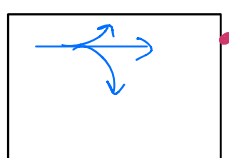
Phase-(III)



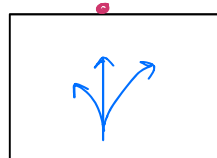
Phase-(IV)

→ Traffic signals.

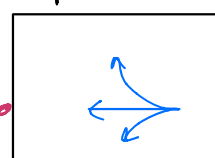
case (II) :-



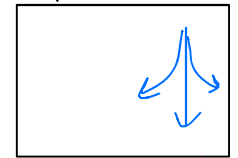
Phase-(I)



Phase-(II)



Phase-(III)



Phase-(IV)

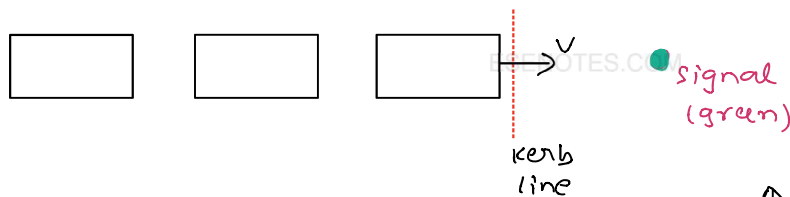
- 5 and 6 phase operation are usually provided if the intersection control is adaptive i.e. signal phase and timing adapt to the real time traffic volume conditions.

(viii) LOSS TIME (t_L) :- Due to traffic signal, traffic streams are continuously started and stopped hence every time this happens a portion of cycle length is not being utilised completely which is termed as loss time. i.e. time which is not serving to the traffic movement.

- Total loss time is combination of

(a) startup loss time (t_{sL}) :- It occurs when a signal indication turns green from red. Drivers in the queue do not instantly start moving and there is an initial lag due to reaction to the change of signal indication.

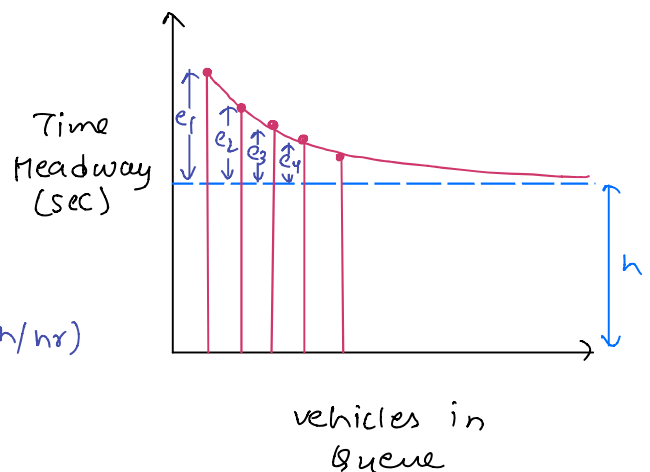
- This startup delay results in a portion of green time not being utilised.



$$t_{sL} = e_1 + e_2 + e_3 + \dots + e_n$$

$$t_{sL} = \sum_{i=1}^n e_i$$

saturation flow rate = $\frac{3600}{h}$ (veh/hr)
of intersection (S_i)
 i = phase number.



- As the signal is initiated, the time interval b/w the two vehicles referred as headway is noted.
- The first headway is the time interval b/w the initiation of green signal and the instant vehicle crosses the kerb line.
- Successive headways are now noted & are observed to be comparatively lower, because the second driver can overlap his reaction time with that

of first driver.

t_{sl} is generally 2sec

- After few vehicles the headway will become constant.

(b) CLEARANCE LOS TIME (t_{cl}) :- when the signal turns yellow from green the later position of time during yellow interval is not utilised by the traffic generally.

- Additionally if there is all red interval, this time period is also generally not utilised by the traffic.

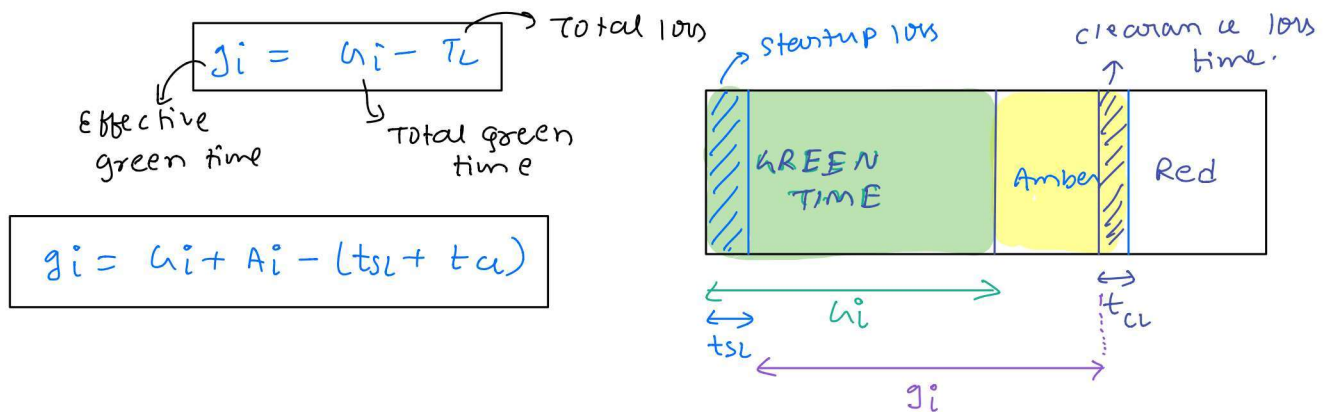
- This period of time during the change and clearance interval that are not effectively utilised is termed as clearance loss time.

NOTE :- In intersection with significant red light running, the clearance loss time may be negligible.

Hence,

$$T_L = t_{sl} + t_{cl}$$

(ix) EFFECTIVE GREEN TIME :- Due to the above loss time, time available for the movement of vehicle in actual is reduced to cross the intersection and is termed as "effective green time".



(x) CAPACITY :- if C = cycle length (sec)

$$\text{no. of cycle in 1hr} = \frac{3600}{C} \quad (\text{cycle/hr})$$

- Effective green time of each cycle = g_i

$$\text{Total eff. green time} = \frac{3600}{C} g_i$$

- If time headway b/w the vehicles is " h " sec

$$\text{Total number of vehicles crossing the intersection in hr.} = \frac{3600}{C} g_i \times \frac{1}{h}$$

$$\text{capacity of lane} = \frac{3600 g_i}{c h} \quad (\text{veh/hr})$$

$$\text{capacity of lane} = s_i \times \text{green ratio}$$

$$s_i = \frac{3600}{h} = \text{saturation flow rate.}$$

$$\frac{g_i}{c} = \text{green ratio.}$$

- 8) cycle time of an intersection is 55 sec. Green time is 25 sec. corresponding yellow time is 3 sec. if the saturation headway is 2.4 sec per vehicle and startup loss time is 2 sec & clearance time is 1 sec. Find the capacity of movement per lane

sol

$$G_i = 25 \text{ sec}$$

$$h = 2.4 \text{ sec.}$$

$$A_i = 3 \text{ sec}$$

$$t_{sl} = 2 \text{ sec}$$

$$t_{cl} = 1 \text{ sec}$$

$$\begin{aligned} \text{capacity} &= \frac{3600 g_i}{h c} \\ &= \frac{3600 \times 25}{2.4 \times 55} = 681.1 \end{aligned}$$

$$\begin{aligned} g_i &= G_i + A_i - 3 \\ &= 25 + 3 - 3 \\ &= 25 \end{aligned}$$

Total time req by n number of vehicles to clear the intersection.

$$T = nh + t_{sl}$$

$$\left\{ \begin{aligned} 25 &= n \times 2.4 + 3 \\ n &= 10 \text{ vehicle} \end{aligned} \right.$$

- (ii) compute the number of vehicle out of 15 at this intersection, that would be pass.

sol 1st Approach :-

$$T = g_i = nh + t_{sl}$$

$$n = \frac{g_i - t_{sl}}{h} = 9.58 = 10 \text{ will pass}$$

$$\text{vehicle that would not able to pass} = 15 - 10 = 5$$

2nd Approach :-

$$\text{Time req to cross 15 veh, } T = 15 \times 2.4 + 2 = 38 \text{ sec} \quad \uparrow t_{sl}$$

$$\text{But time available} = 25 \text{ sec}$$

$$\text{no. of veh. not pass} = \frac{38 - 25}{2.4} = 5.416 \approx 5$$

(XI) CRITICAL LANE VOLUME :-

- During any green signal phase, several lane on one or more move approaches are permitted to move
- One of these will have more traffic volume than the other & will require more time, than any other lane moving at same time.
- If sufficient time is allocated for this lane, than all the other lanes will also be accomodated.
- The volume of this lane is termed as "CRITICAL LANE VOLUME".

Feb-14 # Determination of cycle length on the basis of startup loss time

- let startup loss time be t_{SLi} for i^{th} phase and total number of phases be "n".

$$\text{Total startup loss time for all phases} = \sum_{i=1}^n t_{SLi}$$

$$\text{if } t_{SL} \text{ is const. for all phase} = n t_{SL} = \text{Total startup loss.}$$

- If cycle length be c (sec)

$$\text{No. of cycle per hour} = \frac{3600}{c}$$

$$\therefore \text{Total startup loss time in one hour} = \frac{3600}{c} \times n t_{SL}$$

$$\therefore \text{Remaining time in hour for } (T_g) \text{ movement of the vehicle} = 3600 - \frac{3600}{c} n t_{SL}$$

$$T_g = 3600 \left(1 - \frac{n t_{SL}}{c}\right)$$

$$\text{Now, max sum of critical volume } v_c = \frac{T_g}{h}$$

$$v_c (\text{veh/h}) = \frac{3600}{h} \left(1 - \frac{n t_{SL}}{c}\right)$$

$$v_c = s_i \left(1 - \frac{n t_{SL}}{c}\right)$$

$$\frac{v_c}{s} = 1 - \frac{n t_{SL}}{c}$$

$$1 - \frac{v_c}{s} = \frac{n t_{SL}}{c}$$

cycle time $\leftarrow c = \frac{n t_{sl}}{1 - \frac{v_c}{s}}$ (sec)

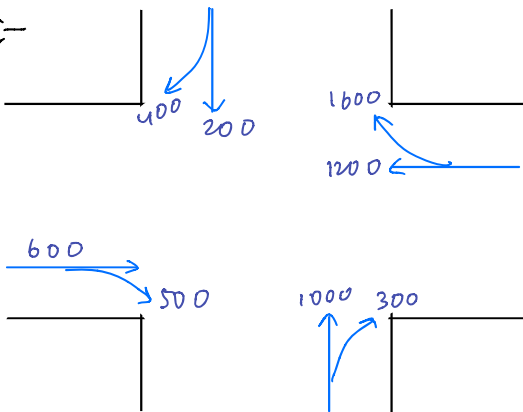
t_{sl} = startup loss time in each phase
 n = number of phase
 v_c = critical volume.
 s = saturation volume
 c = cycle time

NOTE:- If saturation flow for diff phase is different

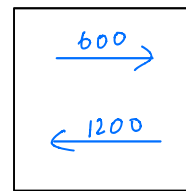
$$c = \frac{n t_{sl}}{1 - \sum \frac{v_{ci}}{s_i}}$$

v_{ci} = critical vol. for i th phase
 s_i = saturation vol. for i th phase

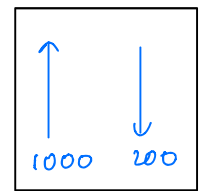
Eg:-



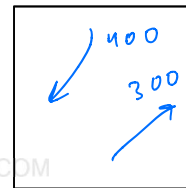
Four phase system



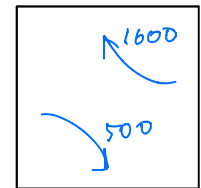
Phase-I



Phase II



Phase III

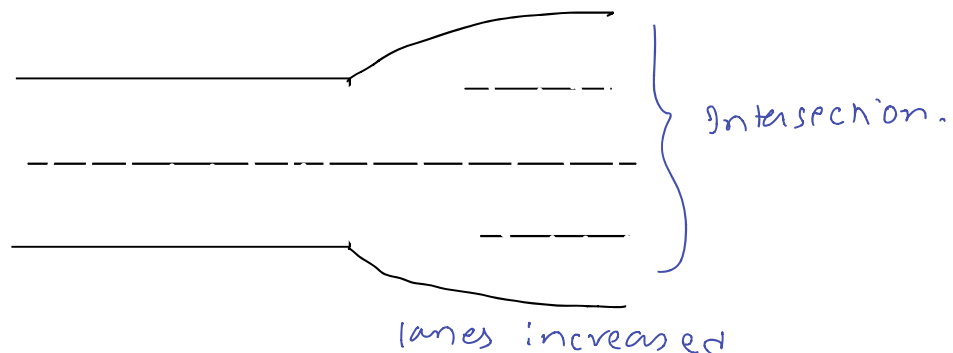


Phase IV

$$\sum \frac{v_{ci}}{s_i} = \frac{1200}{s_1} + \frac{1000}{s_2} + \frac{400}{s_3} + \frac{1600}{s_4}$$

NOTE:- Some time $\sum \frac{v_{ci}}{s_i}$ becomes more than 1 then

c becomes negative which is not possible. Then we reduce v_{ci} by increasing no. of lanes. i.e. suppose $v_{ci} = 1200$ we make 3-lane then v_{ci} become 400. like this we adjust the lanes.



Q) If startup loss time is 3 sec, saturation headway is 2.3 sec for two phase signal compute the cycle length

sol =

$$t_{sl} = 3 \text{ sec}$$

$$h = 2.3 \text{ sec}$$

$$C = \frac{n t_{sl}}{1 - \sum \frac{V_i}{S_i}}$$

$$\text{saturation cap} = \frac{3600}{2.3} = 1566 \text{ veh/hr}$$

$$\sum \frac{V_i}{S_i} = \frac{1800 + 1150}{1566} = 1.885 (>1)$$

cycle time will come out to be negative.

provide 2-lanes for Road (4) and Road (2)
and 2-lanes for Road (1)

$$\therefore \sum \frac{V_i}{S_i} = \frac{\frac{1800}{2} (\text{lane 4}) + 900 (\text{lane 3})}{1566} = 0.941$$

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$$C = \frac{n t_{sl}}{1 - \sum \frac{V_i}{S_i}} = \frac{2 \times 3}{1 - 0.941} = 103.25 \text{ sec.}$$

NOTE:- computation of cycle length is done by assuming the flow of traffic to be uniform over the entire 1 hr duration which is not in actual.

Hence to account for the variation of volume in an hour, a factor termed as peak hourly factor is used.

$$PHF = \frac{\text{hourly volume}}{\text{max. flow rate.}}$$

Hence cycle time,

$$C = \frac{n t_{sl}}{1 - \frac{1}{PHF} \sum \frac{V_i}{S_i}}$$

NOTE (4) :- Highway capacity manual gives the following relationship to compute cycle length.

(***)

$$c = \frac{n L x_c}{x_c - \sum_{i=1}^n \frac{v_i}{s_i}}$$

n = no. of phase

L = loss time per phase

$\frac{v_i}{s_i}$ = volume to saturation flow ratio of i th phase

x_c = Quality factor.

i.e critical vol. capacity ratio.

GREEN SPLITTING :-

Green splitting is the proportioning of effective green time in each of the signal phase.

- Green splitting is done by → for one cycle only

$$g_i = \frac{V_{ci}}{\sum V_{ci}} \times T_g$$

V_{ci} = critical lane volume for i th phase

T_g = Total effective green time of one cycle.

$$T_g = c - \sum t_L$$

$$T_g = c - n t_L$$

c = cycle length

t_L = loss time of i th phase.

$$g_i = A_i + A_i - t_L$$

A_i = Actual green time

for 2-phase

$$T_g = c - 2 t_L$$

$$= g_1 + g_2 - 2 t_L$$

$$= A_1 + A_1 + A_2 + A_2 - 2 t_L \checkmark$$

Q) Phase diagram will flow values of an intersection with two phase is as shown for the loss time and yellow time given for phase-I and phase-II find the green time allocated to each phase. Take cycle length to be 120 sec.

Phase-I

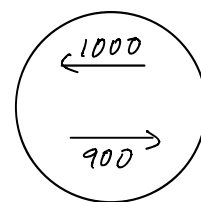
Phase-II

loss time = 2 sec

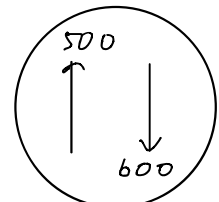
3.5 sec

yellow time = 3 sec

4 sec



phase ①



②

sol

$$c = 120 \text{ sec.}$$

Green splitting phase-I, $g_1 = \frac{V_{c1}}{\sum V_{ci}} \times T_g$

$$T_g = c - n t_L = 120 - (2 + 3.5) = 114.5 \text{ sec.}$$

$$V_{c1} = 1000$$

$$\sum V_{ci} = 1600$$

$$V_{c2} = 600$$

$$g_1 = \frac{1000}{1600} \times 114.5 = 71.56 \text{ sec} \approx 72 \text{ sec}$$

$$g_2 = \frac{600}{1600} \times 114.5 = 42.93 \text{ sec} \approx 42.5 \text{ sec}$$

$$g_1 = G_1 + A_1 - tL_1$$

$$g_2 = G_2 + A_2 - tL_2$$

$$G_1 = 72 - 3 + 2$$

$$= 71 \text{ sec}$$

$$G_2 = 42.5 - 4 + 3.5$$

$$= 42 \text{ sec}$$

0

$G_1 = 71 \text{ sec}$	$A_1 = 3 \text{ sec}$	$R_1 = 46 \text{ sec}$
$R_2 = 74 \text{ sec}$	$G_2 = 42 \text{ sec}$	$A_2 = 4 \text{ sec}$

120

PEDESTRIAN CROSSING REQ :-

- This requirement of pedestrian crossing can be fulfilled by two ways-

(i) By providing the exclusive pedestrian phase. which will increase cycle time.

(ii) When phase is red, pedestrian movement can be allowed in that time.

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Green time for pedestrian = walk time + clearance time

$$G_p = t_w + \frac{w}{v_p}$$

$$v_p = \text{vel. of pedestrian} = 1.2 \text{ m/sec}$$

$$t_w = \text{walk time} = 7 \text{ sec}$$

$$G_{p1} = 7 + \frac{w_1}{1.2} \text{ (sec)}$$

$$G_{p2} = 7 + \frac{w_2}{1.2} \text{ (sec)}$$

Pedestrian can walk as long as or max green time for pedestrian will be the red time for that phase.

$$\max(G_{p1}) = R_1$$

$$\max(G_{p2}) = R_2.$$

SIGNAL DESIGN

Designing of signal can be done as follows :-

(a) WEBSTER METHOD :-

In this method optimum cycle time is computed from least total delay at signalized intersection.

$$\text{optimum cycle time, } C_0 = \frac{1.5L + 5}{1 - Y} \text{ (sec)}$$

$$S_i = \frac{3600}{h} \rightarrow \text{time headway (sec)}$$

$Y \rightarrow$ less than 1

$$L = n t_L + R = \text{loss time}$$

$$n = \text{no. of phases}$$

$$t_L = \text{total loss time} = t_{sL} + t_{cL} \quad (= 2 \text{ sec})$$

$$R = \text{All Red time.}$$

$$Y = Y_1 + Y_2 + \dots + Y_n$$

NOTE :- This method is empirical in nature.

$$Y_1 = q_1/S_1, Y_2 = q_2/S_2, \dots, Y_n = q_n/S_n$$

$q_i =$ critical lane volume per i^{th} phase

$S_i =$ saturation capacity for i^{th} phase.

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Now green time is given by .

$$G_1 = \frac{(C_0 - L) Y_1}{Y_1 + Y_2 + \dots + Y_n}$$

$$G_2 = \frac{(C_0 - L) Y_2}{Y_1 + Y_2 + \dots + Y_n}$$

Feb 15

Q) The avg normal flow of traffic on cross-roads A & B during design period are 400 & 250 PCU per hour. The saturation flow values are 1250 and 1000 pcu/hr respectively. The all red time req. for pedestrian crossing is 12 sec. design two phase traffic signals by webster method. design two phase traffic signal by webster method

SOL

$$\text{optimum cycle time, } C_0 = \frac{1.5L + 5}{1 - Y}$$

$$L = n t_{sL} + R = 12 + 2 \times 2 = 16 \text{ sec}$$

$$Y_1 = \frac{q_1}{S_1} = \frac{400}{1250} = 0.32$$

$$Y_2 = \frac{q_2}{S_2} = \frac{250}{1000} = 0.25$$

$$\left. \begin{array}{l} Y_1 = 0.32 \\ Y_2 = 0.25 \end{array} \right\} Y = Y_1 + Y_2 = 0.57$$

$$C_0 = \frac{1.5 \times 16 + 5}{1 - 0.57} = 67.44 \text{ sec} \approx 67.5 \text{ sec.}$$

$$C_1 = (C_0 - L) \frac{Y_1}{Y} = (67.5 - 16) \times \frac{0.32}{0.57} = 28.91$$

$$C_2 = \frac{0.25}{0.57} (67.5 - 16) = 22.56 \text{ sec}$$

$$C_0 = C_1 + A_1 + R_1$$

$$C_0 = C_1 + A_1 + C_2 + A_2$$

$$A_1 + A_2 = 67.5 - (22.56 + 28.91)$$

$$A_1 + A_2 = 16 \text{ sec (it includes au med)}$$

$$A_1' + A_2' = 16 - R = 16 - 12 = 4 \text{ sec}$$

$$\text{we can say, } A_1' = A_2' = 2 \text{ sec.}$$

Phase diagram

67.5 sec

$C_1 = 28.91 \text{ sec}$	$A_1' = 2 \text{ sec}$	Au med $R = 12 \text{ sec}$	$R_1 = 24.6 \text{ sec}$
$C_2 = 22.56 \text{ sec}$	$A_2' = 2 \text{ sec}$	$R = 12 \text{ sec}$	$R_2 = 24.6 \text{ sec}$

- Q) A fixed time two phase traffic signal is to be designed for an urban intersection using webster's approach. The intersection is having north-south and east-west road where any straight ahead traffic is permitted. The design hourly traffic flow from various arms and the corresponding saturation flow are given as follows. Compute the optimum cycle length by assuming an med period per phase and time lost per phase due to starting delay as 3 sec and 2 sec respectively.

Arm of intersection	North	South	East	West
Design hour traffic flow (Pcu/hr)	1000	600	950	800
Saturation traffic flow (Pcu/hr)	3000	2400	3600	2000

solⁿ $t_L = 2 \text{ sec}$ optimum cycle time $t_0 = \frac{1.5L + 5}{1 - y}$
 $R = 3 \text{ sec.}$

$$L = n t_L + R = 2 \times 2 + 3 \times 2 = 10 \text{ sec}$$

$$y_1 = \max \left\{ \begin{array}{l} \frac{1000}{7000} = 0.33 \\ \frac{600}{2400} = 0.25 \end{array} \right. \quad y_2 = \max \left\{ \begin{array}{l} \frac{950}{3600} = 0.264 \\ \frac{800}{2000} = 0.4 \end{array} \right.$$

$$\therefore y_1 = 0.33, \quad y_2 = 0.4$$

$$y = y_1 + y_2 = 0.73$$

$$t_0 = \frac{1.5 \times 10 + 5}{1 - 0.73} = 75 \text{ sec}$$

(II) ROAD MARKING :-

- marking on the carriageway and on the objects within and adjacent to the road way are used as a means of guiding and controlling the traffic
- They promote road safety and ensure smooth flow of traffic into the required path.
- Carriageway markings are of following types.

(a) A broken line longitudinal line is used for indicating the centre line on two or three lane roads for lane marking on multi-lane roads.

- Drivers may cross these at their discretion, if traffic permits

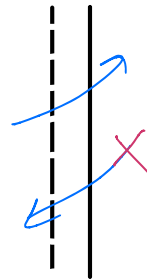
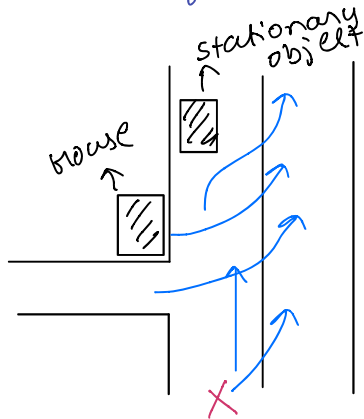
(b) Longitudinal solid lines are used as guiding or regulating lines and are not meant to be crossed by the driver, except for entry or exit from a premises or a side road or to avoid a stationary object.

(c) Double solid line indicates maximum restriction and are not to be crossed except in emergency.

(d) In a combination of broken and solid lines, a solid line may be crossed with discretion of the driver, closer/nearer to it. But solid line cannot

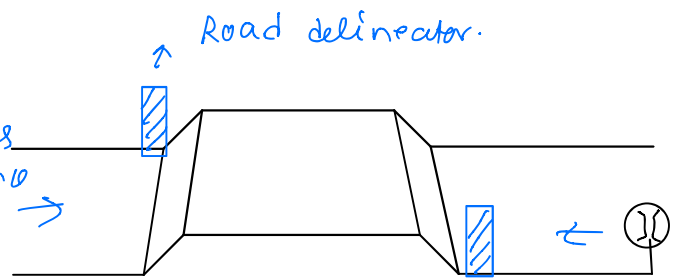
be crossed.

- (e) solid lines either parallel to the intersecting roadway or at right angles to the direction of approaching traffic mark the position of stop line before road intersection.



(III) ROAD DELINEATORS :-

- The purpose of delineator is to provide visual assistance to driver about the alignment of the road ahead especially at the night.



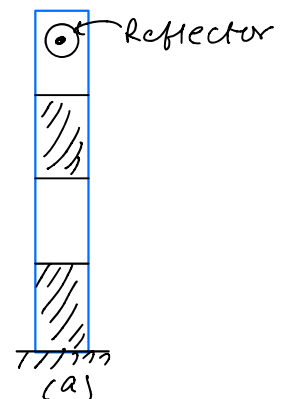
- Reflectors are used on delineators for better night visibility.

- These are further classified as:-

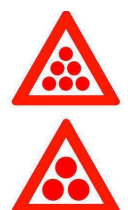
(a) Road way indicator :- This are intended to delineate the edges of the roadway so as to guide alignment of the road ahead.

(b) Hazard marker :- This are to define obstruction, like guard-rails, abutments, culvert etc which are narrower than roadway width at approaches.

(c) object marker :- These are used to indicate the hazard and obstruction within the vehicle flow path
eg:- channelizing islands.



HAZARD MARKER



Object Marker

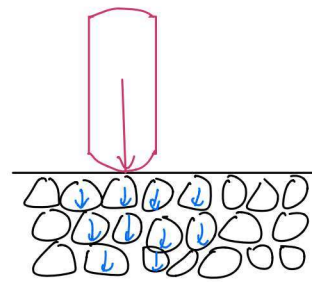
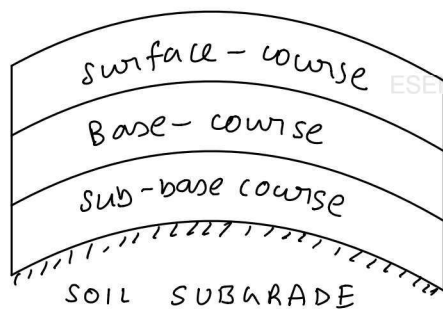
(Q) TRAFFIC SIGN :-

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PAVEMENT DESIGN

- A pavement is a load bearing and load distributing component of a road.
- For geometric design we need to consider all types of vehicles but for pavement design only vehicles having significant heavy loads are considered.
- These vehicles are generally commercial vehicles
- As per IRC vehicles having gross load > 3 tonnes are called commercial vehicles.
- Pavement can be classified as follows :-
 - (a) Flexible.
 - (b) Rigid.
 - (c) Semi-rigid.
 - (d) Composite.

(a) FLEXIBLE PAVEMENT :-

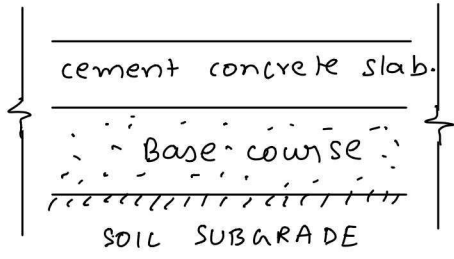


grain to grain contact.

- A flexible pavement is one that is made up of one or more layers of material, the highest quality material forming the top layer.
- The load carrying capacity of the flexible pavement is derived from the load distribution property and not from its flexural strength.
- The flexible pavement layer reflects the deformation of the lower layer. Thus if lower layer of the pavement or soil subgrade is undulated the pavement surface also gets undulated.
- This pavement transmits its load or vertical stress to the lower layer by grain to grain contact.
- Major pavement failure are FATIGUE, CRACKING & RUTTING, THERMAL CRACKING (*) IRC (*) IRC

(Rutting is depression in localised area).

Feb 16 (b) RIGID PAVEMENT :-

- Rigid pavement is constructed with cement concrete slabs.
 - A major contributor to the load bearing capacity is the slab itself as it derives this property by virtue of its flexure strength.
- 
- The diagram illustrates the vertical structure of a rigid pavement. It consists of three distinct layers. The top layer is a solid horizontal line labeled 'cement concrete slab'. Below this is a layer with a dashed, stippled pattern labeled 'Base course'. The bottom layer is a horizontal line with diagonal hatching below it, labeled 'SOIL SUBGRADE'. Brackets on the left and right sides of the 'Base course' and 'SOIL SUBGRADE' layers indicate their relative positions within the pavement structure.
- Rigid pavement layer doesn't reflect the deformation of the lower layer due to the presence of concrete slab at the top.
 - Here the transfer of the load is due to layer to layer contact.
 - Here the major mode of failure are IRC consider this **FATIGUE, RUTTING** and **PUMPING**
- { Here pumping is the ejection of soil slurry through joints and cracks of cement concrete pavement causing downward movement of slab under heavy load. }

NOTE:- The initial cost of rigid pavement is more than flexible pavement whereas the maintenance cost of flexible is more than rigid pavement.

- 2) Expansion and contraction joints are required in rigid pavement, but not in flexible pavement.

(c) Semi-Rigid Pavement:-

- This pavement represents the intermediate stage b/w the flexible and rigid type.
- It derives the strength by both load spreading as well as flexure action.
- In this case bonded material like pozzolona eg- fly ash, lean cement concrete, or soil cement are used in base course or sub-base course which imparts flexural strength to it.

(d) Composite Pavement :- It is a mixture of above types in layers.

- For eg pavement consisting of lean concrete base, a roller compacted concrete slab over it and a surfacing of bituminous concrete.

FUNCTIONS OF PAVEMENT COMPONENT :-

(i) SOIL SUBGRADE :- Its function is to ultimately carry the vehicular load and to drain off the water seeping from the surface.

- Top 50cm layer of soil subgrade is compacted properly.

(ii) SUB-BASE & BASE COURSE :- These layers are composed of broken stone aggregates of smaller size.

- Base and sub-base course are used in flexible pavement for increasing the load carrying capacity by distribution.

- Base course is provided in rigid pavement for preventing pumping and protecting the subgrade against FROST ACTION.

(iii) BEARING COURSE :- Purpose of this course is to give smooth riding surface to the vehicles.

- It resists pressure exerted by vehicles and is subjected to wear and tear.
- It offers water tightness to the pavement.

DESIGN CONCEPTS OF PAVEMENT :-

(i) LOAD TRANSFER THROUGH WHEELS :-

- While analysing load transfer through wheels we need to consider total wheel load and distribution of pressure.
- Pressure generated at the point of contact b/w tyre and pavement is termed as contact pressure, where as pressure inside the tyre is termed as "TYRE PRESSURE".

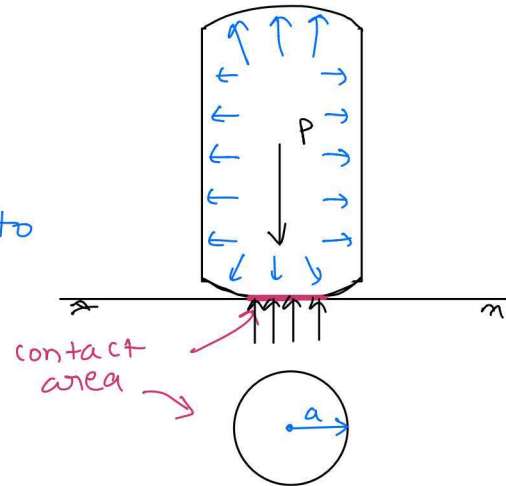
$$\text{contact pressure} = \frac{\text{wheel load}}{\text{contact area}}$$

$$= P/A$$

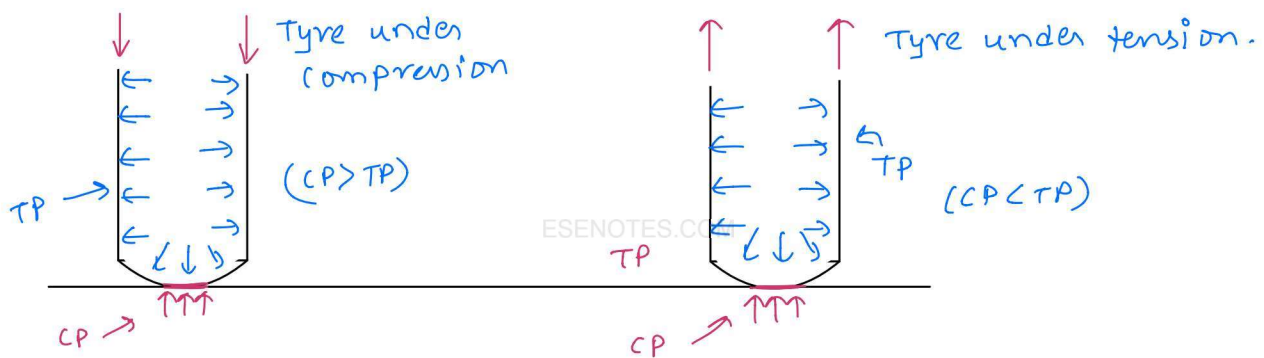
If contact area is assumed to be circular then,

$$A = \pi a^2$$

$$\text{Tyre pressure} = 7 \text{ kg/cm}^2$$



- At low tyre pressure, tyre comes under compression and contact pressure $>$ tyre pressure.
- At high tyre pressure, the tyre comes under tension or contact pressure $<$ tyre pressure.



(ii) RIGIDITY FACTOR :- For designing these pressures are related with Rigidity factor.

$$\text{Rigidity Factor} = \frac{\text{Contact Pressure}}{\text{Tyre Pressure}}$$

For designing $CP = TP = 7 \text{ kg/cm}^2$ and $RF = 1$.

$$\text{if } TP > 7 \text{ kg/cm}^2, \quad RF < 1$$

$$TP < 7 \text{ kg/cm}^2, \quad RF > 1$$

- For designing of pavement, design load is computed by considering following factors.

- The traffic volume will increase with time.
- Different vehicles will have diff wt.
- wheel loads will be dispersed across the pavement

(a) This factor is accounted by designing the pavement for future traffic (i.e. at the end of design life).

$$N = \frac{365 A [(1+r)^n - 1]}{r}$$

N = cumulative number of commercial vehicles at the end of design life.

A = initial design traffic in the year of completion of construction.

$$A = P (1+r)^x$$

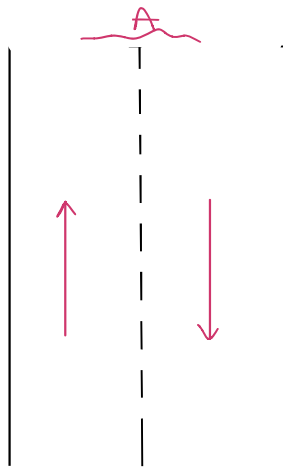
P = No. of commercial vehicle / day at last count

r = rate of increase of traffic.

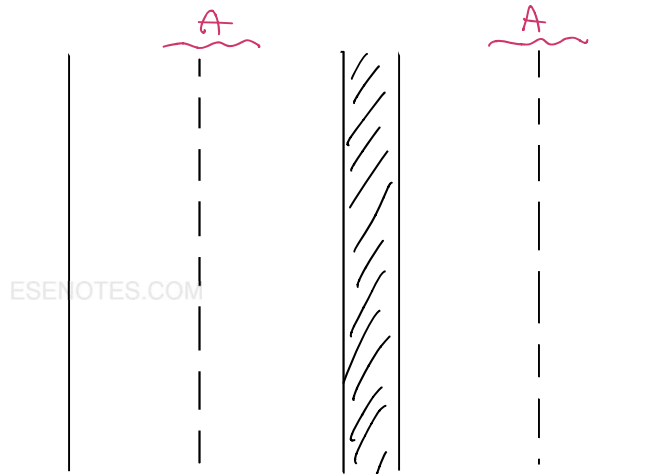
x = construction period in years.

n = Design life of pavement.

A is calculated per carriage way which included traffic in both directions.



2-lane 2-way single carriage way.

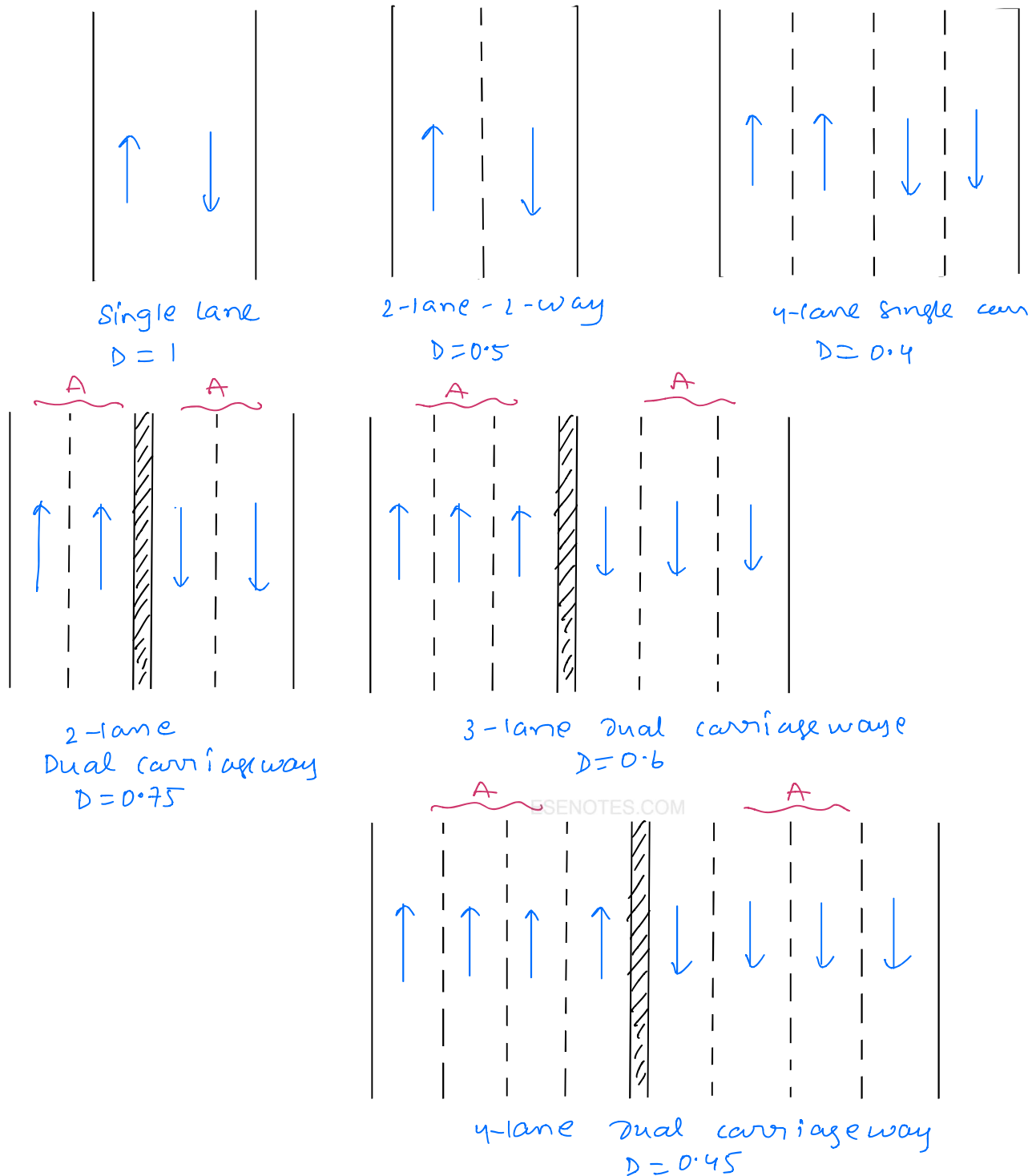


Dual carriage way

(c) This is accounted by using a factor which distributes the commercial traffic in each direction on each lane and is termed as "LANE DISTRIBUTION FACTOR" / "LATERAL OR DIRECTIONAL DISTRIBUTION FACTOR".

As per IRC 37-2018

Road	<u>D</u>
(i) Single lane road.	1
(ii) Intermediate lane road.	0.75
(iii) Two lane two way single carriage way road	0.5
(iv) 4 lane single carriage way road.	0.4
(v) Dual carriage way two lane -	0.75
(vi) Dual carriage way 3-lane road.	0.6
(vii) Dual carriage way 4-lane road	0.45



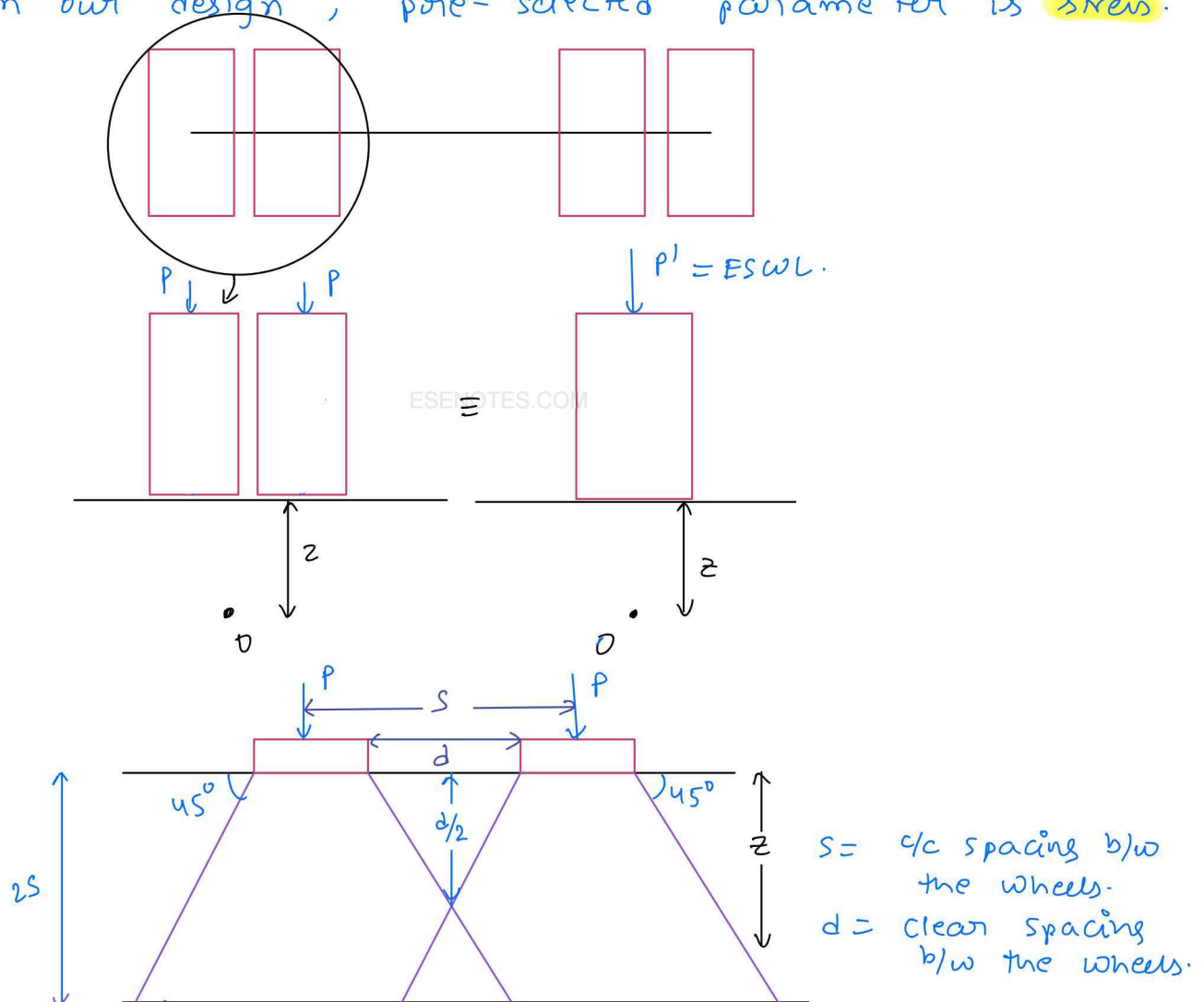
Feb 17 (b) Traffic load consideration in design.

- In pavement design, traffic load consideration is done by following approaches.

(i) FIXED TRAFFIC APPROACH :-

- In this heaviest vehicle is considered.
- Here number of repetitions are not considered.
- Pavement is designed on the basis of single wheel load.

- Multiple wheel loads are converted into "EQUIVALENT SINGLE WHEEL LOAD" (ESWL).
- This approach is used for designing of airport pavement as here single load movement is sufficient to cause the damage.
- ESWL is defined as load on single tyre/wheel which will cause an equivalent magnitude of pre-selected parameter [stress, strain, deflection, etc] at a given location to that resulting from multiple wheel load at same location.
- In our design, pre-selected parameter is stress.



For $z < d/2 \rightarrow \text{ESWL} = P \rightarrow$ No stress overlap.

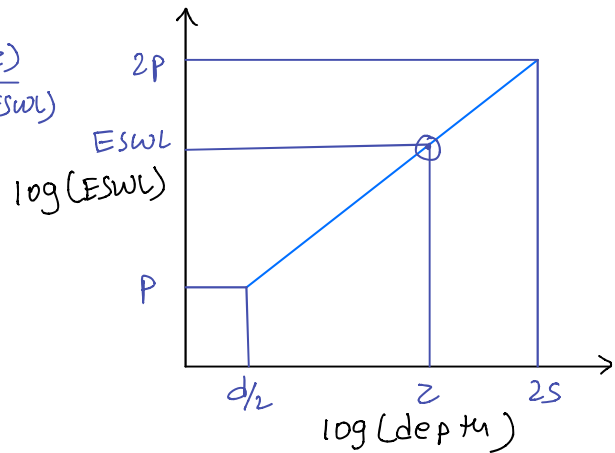
For $z > 2S \rightarrow \text{ESWL} = 2P \rightarrow$ Full overlap of stress is considered.

$\frac{d}{2} < z < 2S \rightarrow$ ESWL is computed by interpolation considering the variation of ESWL to be linear with depth on log scale.

$$\frac{\log(2S) - \log(d/2)}{\log(2P) - \log(P)} = \frac{\log(2S) - \log(z)}{\log(2P) - \log(ESWL)}$$

$$\frac{\log\left(\frac{4S}{d}\right)}{\log(2)} = \frac{\log\left(\frac{2S}{z}\right)}{\log\left(\frac{2P}{ES}\right)}$$

$$\log\left(\frac{2P}{E}\right) = \frac{\log\left(\frac{2S}{z}\right) \log(2)}{\log(4S/d)}$$



- 8) calculate the ESWL of dual wheel assembly carrying 2050 kg each for pavement thickness of 5, 40 and 65 cm. c/c spacing b/w tyre is 30 cm and distance b/w adjacent wheel of tyre is 20 cm.

Sol $S = 30 \text{ cm}$, $d = 20 \text{ cm}$.

For pavement thickness, $z = 5 \text{ cm}$.

$$z < d/2$$

$$ESWL = P = 2050 \text{ kg.}$$

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For pavement thickness, $z = 40 \text{ cm}$

$$\frac{d}{2} < z < 2S$$

$$\frac{\log(60) - \log(10)}{\log(4100) - \log(2050)} = \frac{\log(60) - \log(40)}{\log(4100) - \log(E)}$$

$$\frac{\log 6}{\log 2} = \frac{\log 1.5}{\log\left(\frac{4100}{E}\right)}$$

$$\log\left(\frac{4100}{E}\right) = 0.0681$$

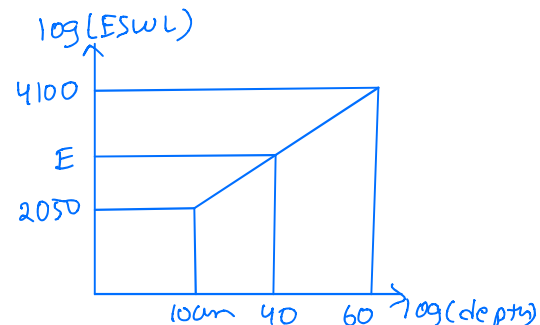
$$\frac{4100}{E} = 1.169$$

$$ESWL = 3504.8 \text{ kg.}$$

For pavement thickness $z = 65$

$$z > 2S$$

$$ESWL = 2P = 4100$$



(ii) FIXED VEHICLE APPROACH :-

- In this case design is governed by number of repetition of standard axle.
- 80 kN single axle load is considered to be standard axle load.
- Axle that are not either single or not equal to 80 kN are converted to equivalent single axle load using a factor called Equivalent Axle load factor (EALF).
- EALF hence defines the damage caused to the pavement by one application of the axle load under consideration relative to the damage caused by single application of a standard axle of 80 kN load.
- EALF is computed as follows

$$EALF = \left(\frac{\text{Axle load}}{\text{Standard axle load}} \right)^4$$

- we can multiply the repetition of given axle load by EALF to obtain Equivalent number of repetition of 80 kN axle load.
- Sum of the equivalent number of repetition obtained from all the axle load during design life is used for designing of pavement.
- This approach is used for designing of highway.

$$\text{Total no. of repetitions of equivalent standard axle load} = \sum_{i=1}^m (EALF)_i \cdot n_i$$

n = number of axle load group

$EALF_i$ = Equivalent axle load factor for i^{th} group.

n_i = number of repetition of axle load of i^{th} group.

- with the help of total number of repetition of standard axle a parameter termed as vehicle damage factor (F) can be computed.

- VDF (F) is equivalent to number of standard axle per commercial vehicle.

Q) i) The result of the one day axle load survey of truck on a road is tabulated as below. Find the number of repetition of a standard 80 kN axle in a year.

(ii) If 950 number of axles were surveyed and the no. of vehicles were 400. Find the VDF.

wt (kN) 0-40 40-80 80-120 120-160

Frequency 50 250 400 25

weight (kN)	frequency (n)	Average wt (kN)	$EALF = \left(\frac{Avg\ w}{80}\right)^4$	$EALF \times n_i$
0-40	50	20	0.0039	$0.0039 \times 50 = 0.195$
40-80	250	60	0.316	79
80-120	400	100	2.44	976.4
120-160	25	140	9.37	234.25

Total no. of repetition of standard axle = 1289.8

Total no. of repetition of std. axle load for one year = 1289.8×365
= 470793.425 times.

1289.8 damage is put by 80kN std. axle which is equivalent of 400 vehicle travelling one day.

$\therefore$ for one vehicle, the damage would be = $\frac{1289.8}{400}$

VDF = 3.25

Feb 19 Q) Number of commercial vehicles per day when construction is completed is 5640.

Design life = 20 years.

Rate of growth of traffic = 7.5%

load safety factor = 1.3, lane distribution factor = 0.75

Find out design traffic in msa for the following data

Axle (tonnes)	18	14	10	8	6
Frequency (%)	10	20	35	15	20

Sol

$$N = \frac{365 A [(1+r)^n - 1]}{r}$$

where, $A = 5840$, $n = 20 \text{ yrs}$, $r = 7.5\%$

$$N = \frac{365 \times 5840 [(1 + 0.075)^{20} - 1]}{0.075} =$$

$$N = 89147017$$

Lane distribution factor = 0.75

$$\therefore \text{Design traffic} = 0.75 \times 89147017 = 66860262.75$$

If damaging potential of traffic is to be considered

No. of std axle in design. = Design traffic $\times$ VDF.

$$\% f = \frac{n_i}{n}$$

$$18 \text{ ton} = \frac{18 \times 10^4}{102} \text{ kN}$$

$$VDF = \sum_{i=1}^m f_i \times EALF_i = 0.1 \times \left(\frac{180}{80}\right)^4 + 0.2 \left(\frac{140}{80}\right)^4 + 0.35 \left(\frac{100}{80}\right)^4 + 0.15 \left(\frac{80}{80}\right)^4 + 0.2 \times \left(\frac{6}{20}\right)^4$$

$$VDF = 5.496$$

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No. of standard axle in Design life = Design traffic $\times$ VDF $\times$ Load Safety

$$= 66860262.75 \times 5.496 \times 1.3$$

$$= 477.7 \times 10^6 \text{ standard axle}$$

$$= \frac{477.7 \times 10^6}{10^6} \text{ msa}$$

$$= 477.7 \text{ msa.}$$

NOTE :- cumulative number of standard axle repetition throughout the design life will be

$$N_s = \frac{365 A [(1+r)^n - 1]}{r \times 10^6} \times LDF \times VDF \times LSF \text{ msa.}$$

million standard
axle. $\leftarrow$

$\rightarrow$ cumulative
msa = 10^6 (csa) standard axle

DESIGN OF FLEXIBLE PAVEMENT :-

Designing of pavement can be classified into following

(i) Empirical Method :-

- This are based on physical properties and strength parameter of soil subgrade.
- GROUP INDEX MTD, CBR METHOD, STABILOMETER MTD, MC-LEAD MTD are empirical in approach

(ii) Semi-Empirical Mtd :-

- These are based on stress-strain method and observation
- TRIAXIAL MTD.

(iii) THEORETICAL MTD :-

- This are based on mathematical computations
- BURMISTER MTD.

(i) GROUP INDEX MTD :-

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This method is based on index property of soil i.e. the properties which helps in identification and classification of soil.

- A characteristics group index is used to indicate the performance of the soil when used as pavement material.
- GI varies from 0-20
- Higher the GI poorer is the performance of soil when used as pavement material, hence higher thickness of pavement is required.

$$GI = 0.2a + 0.005ac + 0.01bd.$$

$a = \% \text{ passing thru } 75\mu \text{ sieve } (0 < a < 40)$

$a = p - 35$ $b = \% \text{ particles passing thru } 75\mu.$

$b = p - 15 \text{ } \nless 40$

$c = w_L - 40 \text{ } \nless 20$ $[0-20]$ $w_L = \text{liquid limit}$

$d = I_p - 10 \text{ } \nless 20$ $[0-20]$ $I_p = \text{plasticity index.}$

- To design the thickness by this method first AI of soil is found.

- Anticipated (expected) traffic is estimated and is designated as follows.

- Traffic volume no. of veh/day.

light	<50
medium	50-300
heavy	>300

- Based on the anticipated traffic an AI value, thickness of pavement layer is found.

- The thickness of sub-base depends on only AI
The " " BASE & surface course is dependent on both AI and Traffic

- value of thickness of different layer of pavement is directly read from either table / curve

SURFACE	} $AI + \text{Traffic}$
BASE	
SUB BASE	} AI
SUBGRADE	

Drawback :-

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- This method does not consider quality of material used for pavement construction.

- hence thickness pavement come out to be same for good & poor quality material.

Q) A soil subgrade sample collected from site was analysed and the results obtained are as follows.

(i) soil proportion passing 0.075 mm sieve = 60%

(ii) $w_L = 45\%$

(iii) $w_p = 23\%$

Determine the AI of subgrade and design pavement thickness using AI . Use the following data,

AI values	0	5	10	15	20
Total thickness (cm)	22	35	43	48	52

$$a = p - 35 = 60 - 35 = 25$$

$$b = p - 15 = 60 - 15 = 45 \quad (>40) = 40$$

$$c = w_L - 40 = 45 - 40 = 5$$

$$d = I_p - 10 = 45 - 23 - 10 = 12$$

$$\begin{aligned}
 hI &= 0.2a + 0.005ac + 0.01bd \\
 &= 0.2 \times 25 + 0.005 \times 25 \times 5 + 0.01 \times 40 \times 12 \\
 hI &= 10.425
 \end{aligned}$$

$$\text{Thickness} = 43.42 \text{ cm.}$$

(ii) CBR METHOD :-

- It is based on strength parameter of subgrade soil and subsequent pavement material.
- In order to design a pavement by CBR method, first soaked CBR value of soil subgrade is evaluated and the appropriate design curve is chosen by taking the design wheel load or by anticipated traffic into consideration.
- The total thickness of pavement needed to cover the subgrade of known CBR value is obtained.
- In case there is a material superior than soil subgrade such that it may be used as sub-base or base course thickness of constructions over this material could also be obtained from the curve.

Feb 20 - Based on CBR value on any material, over which a flexible pavement is required, thickness of the pavement over this is given by.

$$t = \sqrt{P} \left[\frac{1.75}{\text{CBR}} - \frac{1}{k\pi} \right]^{1/2}$$

$$= \left[\frac{1.75P}{\text{CBR}} - \frac{P}{k\pi} \right]^{1/2}$$

$$t = \left[\frac{1.75P}{\text{CBR}} - \frac{A}{\pi} \right]^{1/2}$$

t = thickness (cm)

P = wheel load (kg)

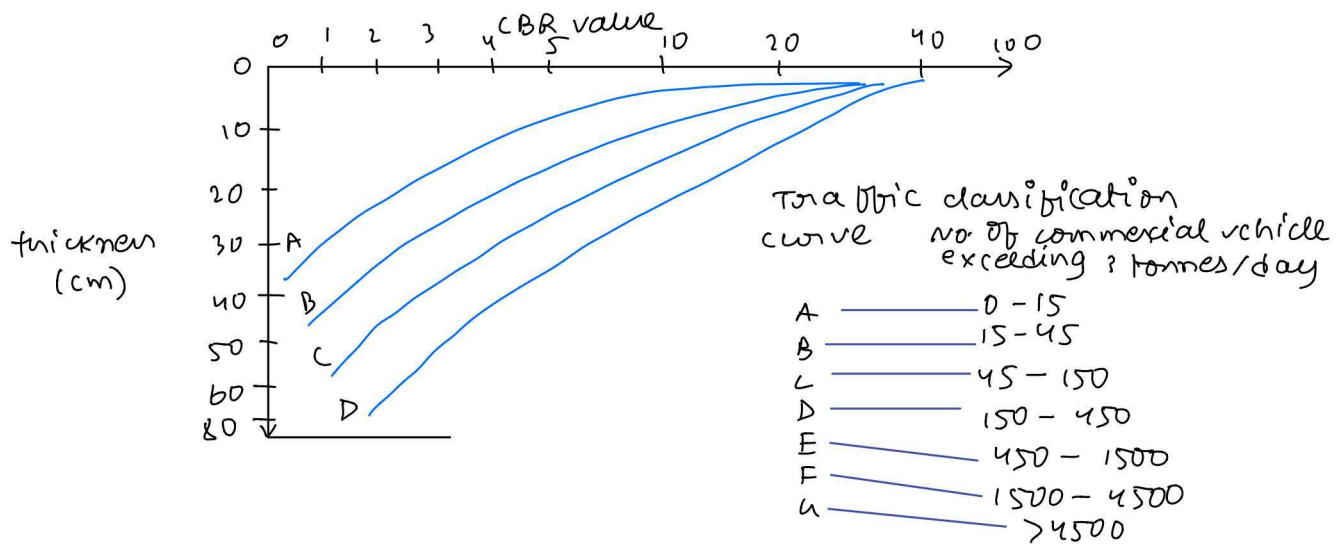
CBR = California Bearing ratio (%)

P = Tyre pressure (kg/cm^2)

A = Area of contact (cm^2)

NOTE:- (i) This expression is only valid when CBR values of the subgrade soil is less than 12%.

(ii) The thickness by this mtd can also be read from the Standard curves.



NOTE :- 1) To use this curves no. of heavy vehicles / day for design is required which as per IRC is computed as follows -

$$A = P(1+r)^{n+10}$$

A = no. of heavy vehicles/day for design.

P = no. of heavy vehicles/day at last count.

r = annual rate of inc of traffic (7.5%)

n = number of years b/w last count and the year of completion.

2) i) CBR is used to determine the material property for pavement design.

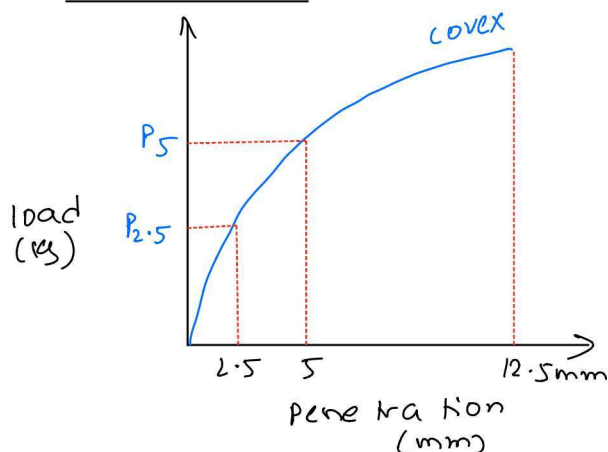
ii) In this test a standard piston of dia 50mm ($A = 19.62 \text{ cm}^2$) is used to penetrate the soil at rate of 1.25 mm/min upto 12.5 mm

iii) Load req by piston to penetrate upto the depth of 2.5 mm and 5 mm is noted and is compared with that load req by the piston to penetrate into the std sample of crushed aggregate

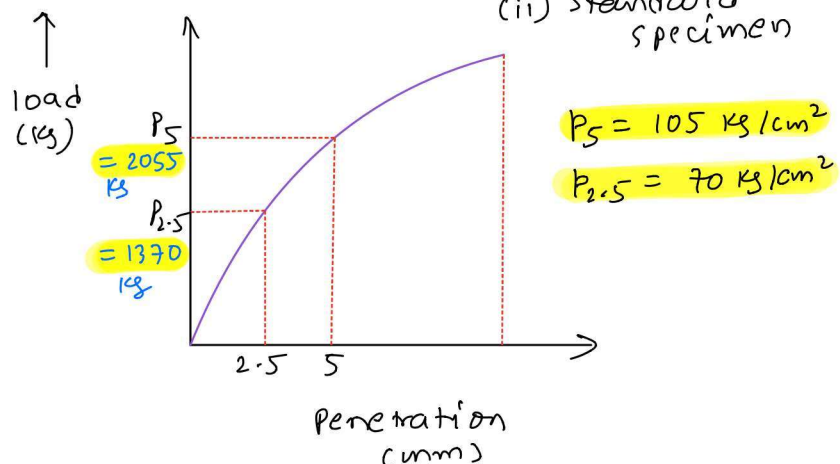
iv) CBR is given by -

$$\text{CBR} = \frac{\text{Load carried by test specimen} -}{\text{Load carried by std. specimen}} = \frac{(P_t)}{(P_s)}$$

(i) Test specimen



(ii) Standard specimen



(v) In most cases CBR decreases with penetration.

$$CBR_{2.5} > CBR_5$$

(vi) If CBR_5 is found more than $CBR_{2.5}$ then test should be repeated and after repetition of that test whichever will come report it as CBR value.

(vii) This mtd gives total thickness requirement of the pavement above a subgrade and this is independent of quality of material used.

8) Soil subgrade sample was obtained from the project site and the CBR test was conducted at field density.

- It is desired to use the following material for diff pavement layer.

(i) compacted sandy soil with 7% CBR.

(ii) Poorly graded gravel with 20% CBR.

(iii) well graded gravel with 95% CBR.

(iv) Thickness of bituminous concrete surface is 5cm.

The traffic survey revealed the present ADT of commercial vehicle as 1200. The annual rate of growth of traffic is found to be 8%. The pavement construction is to be completed in 3 years after the last traffic count. Use the following data,

Penetration (mm)	0	0.5	1	1.5	2	2.5	3	4	5	7.5	10	12.5
Load (kg)	0	5	16.2	28	40	48.5	56	67	75	89	99	106

Due to the surface irregularities a correction of 10% and 5% is to be applied for 2.5 and 5mm penetration.

Design the pavement section by CBR method as per IRC using all the four pavement materials.

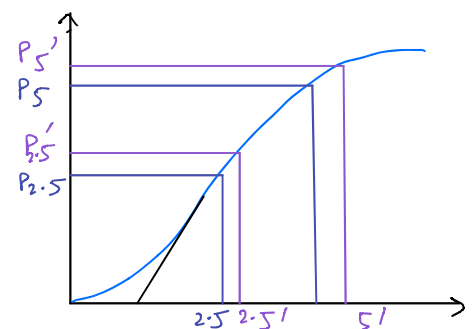
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=

$$P'_{2.5} = 1.1 \times 48.5 = 53.35 \text{ kg}$$

$$P'_5 = 1.05 \times 95 = 98.75 \text{ kg}$$

$$CBR_{2.5} = \frac{53.35}{1370} \times 100 = 3.89\%$$

$$CBR_5 = \frac{98.75}{2055} \times 100 = 3.83\%$$



Adoptn $CBR = CBR_{2.5} = 3.89\%$

$$A = P(1+r)^n = 1200(1+r)^{n+10}$$

$$= 1200(1+0.08)^{3+10} = 3263.54$$

Traffic falls in curve "F". From curve F, corresponding to $CBR = 3.89$ the thickness = 55 cm.

Total thickness above soil subgrade = 55 cm.

(i) for sandy soil with 7% CBR.

From the same curve, thickness $t_1 = 40$ cm.

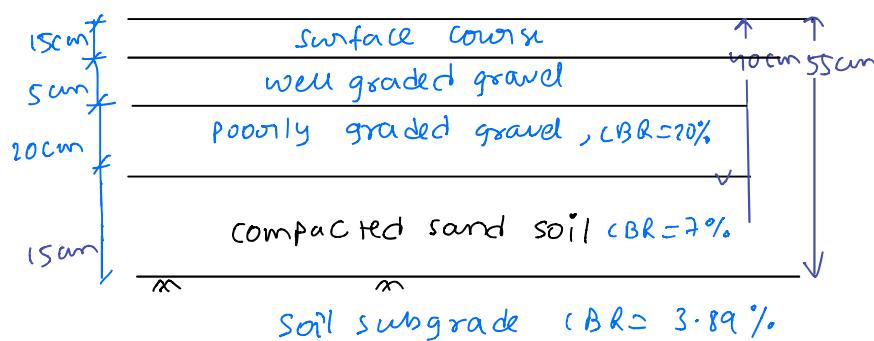
$$\therefore \text{thickness of sub-base} = 55 - 40 = 15 \text{ cm}$$

(ii) for $CBR = 20\%$, thickness, $t_{\text{Base course}} = 20$ cm

$$\therefore \text{thickness of base-course} = 40 - 20 = 20 \text{ cm}$$

(iii) for $CBR = 95\%$, $t_4 = 15$ cm.

$$\therefore \text{thickness of surface course} = 20 - 15 = 5 \text{ cm}$$



(iii) TRIAXIAL TEST METHOD :-

- The pavement thickness T_s consisting of material with E_s is given by

$$(cm) \leftarrow T_s = \sqrt{\left(\frac{3PY}{2\pi E_s \Delta}\right)^2 - a^2}$$

P = wheel load (kg)
 X = Traffic coeff.
 Y = Rainfall coeff

E_s = Modulus of elasticity.
 Δ = Design deflection (0.25 cm)
 a = radius of contact area (cm)

- If pavement and subgrade are considered as a two layer system a stiffness factor has to be introduced to take into account the quality of pavement material in terms of its mod. of Elasticity.

In such case thickness is given by.

$$T_P = \left[\sqrt{\left(\frac{3PY}{2\pi E_S \Delta} \right)^2 - a^2} \right] \left[\left(\frac{E_S}{E_P} \right)^{1/3} \right]$$

here stiffness factor = $\left(\frac{E_S}{E_P} \right)^{1/3}$

E_P = Mod. of elasticity of pavement.

NOTE:- The relation b/w pavement layers of thickness t_1 & t_2 of modulus of elasticity is

$$\frac{t_1}{t_2} = \left(\frac{E_2}{E_1} \right)^{1/3}$$

- Q) Design a pavement section by triaxial method using the following data.

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wheel load = 4050 kg

Radius of contact area = 15 cm.

Traffic coeff = 1.6

Rainfall coeff = 0.7

Design deflection = 0.25 cm

$E_S = 120 \text{ kg/cm}^2$

$E_{\text{Base course}} = 360 \text{ kg/cm}^2$

$E_{\text{bituminous}} = 1200 \text{ kg/cm}^2$

Thick of bit. layer = 7 cm.

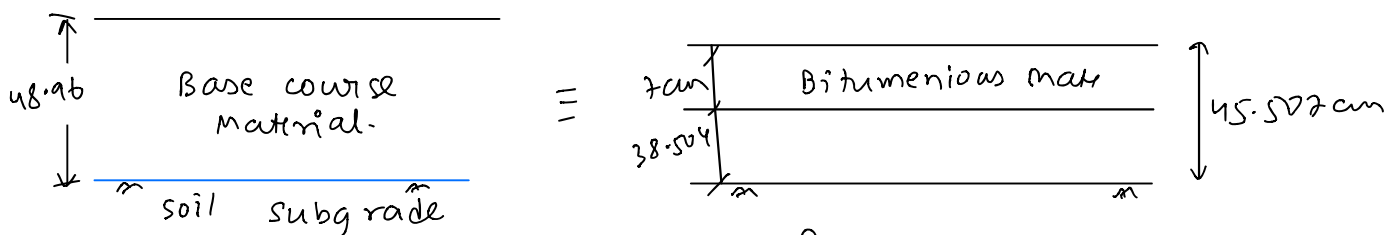
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Thickness of Base course

$$T_P = \left[\sqrt{\left(\frac{3PY}{2\pi E_S \Delta} \right)^2 - a^2} \right] \left[\frac{E_S}{E_P} \right]^{1/3}$$

$$T_P = \left[\sqrt{\frac{3 \times 4050 \times 1.6 \times 0.7}{2\pi \times 120 \times 0.25}} \right] \left[\frac{120}{360} \right] = 48.96 \text{ cm}$$

Thickness of base course = 48.96 cm.



$$T_{Bt} = 7 \text{ cm}, \quad E_{Bt} = 1200 \text{ kg/cm}^2$$

$$T_B = 48.96, \quad E = 360$$

$$\left(\frac{\tau_B}{\tau_{Bt}}\right) = \left(\frac{E_{0t}}{E_B}\right)^{1/3}$$

$\tau_B = 10.456 \text{ cm}^2 \rightarrow$ equivalent of 7cm Bituminous material.

Remaining thickness of base course after providing bituminous layer
 $= 48.96 - 10.456$
 $= 38.504 \text{ cm}.$

Feb 22

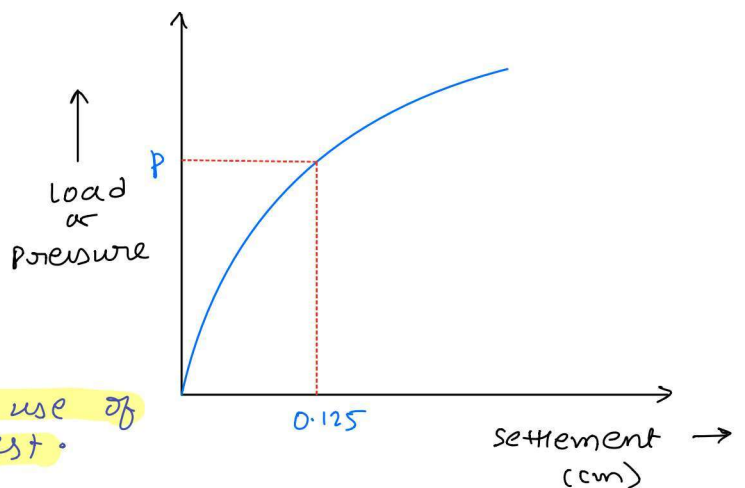
DESIGN OF RIGID PAVEMENT :-

- It is constructed using cement concrete and its load carrying capacity is primarily due to the rigidity of the slab.
- Rigidity is due to high modulus of elasticity of concrete.
- cement concrete pavement is rested on soil foundation which can be treated as a spring having spring constant "K",
- This "K" is termed as modulus of subgrade reaction.
- K is defined as the load or pressure required to attain standard settlement of (12.5 mm) and is found using plate load test.

$$K = \frac{P}{\Delta} \text{ (kg/cm}^2\text{)}$$

- To perform the plate load test different size of plates are available 75cm, 60cm, 45cm, 30 cm.

However IRC recommends the use of 75cm plate for this test.



- As per IRC the modulus of sub-grade reaction corresponding to 75cm plate used for testing, is taken half of that corresponding to 30cm plate.

$$K_{75\text{cm}} = 0.5 K_{30\text{cm}}$$

NOTE:- For plate load test, $\Delta = 1.18 \frac{Pa}{Es}$ - (i), $\Delta = \frac{P}{K}$ - (ii)

From (i) & (ii)

$$\frac{1.18 \text{ Pa}}{E_s} = \frac{b}{K}$$

$$K a = \frac{E_s}{1.18}$$

$K a = \text{constant}$

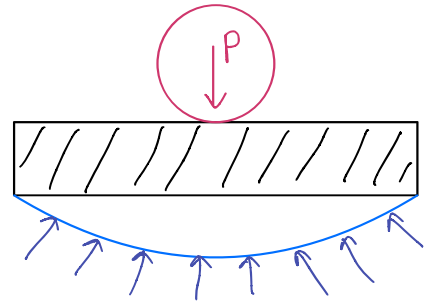
Hence the value of "K" depends on size (a) of the plate

$$\Rightarrow K_{75} (75) = K_{30} (30)$$

$$K_{75} = 0.4 K_{30} \rightarrow \text{But IRC suggest to take } K_{75} = 0.5 K_{30}$$

RADIUS OF RELATIVE STIFFNESS :- It is a radius of area of soil effective in resisting the deformation of slab/pavement

$$l (\text{cm}) = \left[\frac{E h^3}{12 K (1 - \mu^2)} \right]^{1/4}$$



E = Mod. of elasticity of concrete (kg/cm^2).

$$E = (3 \times 10^5 \text{ kg/cm}^2)$$

μ = Poisson's ratio = 0.15

h = Thickness of slab (cm)

K = modulus of subgrade rxn (kg/cm^3)

NOTE :- (i) Only a small % of area of pavement offers the resistance to the load by the virtue of its moment carrying capacity.

(ii) The radius of this area is termed as equivalent radius of resisting section and is given by "Westergaards"

$$b (\text{cm}) = \begin{cases} \frac{1}{4} \cdot \sqrt{1.6 a^2 + h^2} - 0.675 h, & a < 1.724 h \\ a & \text{otherwise} \end{cases}$$

h = slab thickness (cm)

a = radius of contact area (cm).

8) (a) compute the radius of relative stiffness of 15cm thick cement concrete slab, from the following data.

$$E_c = 21 \times 10^4 \text{ kg/cm}^2$$

$$\mu = 0.13$$

$$K = 3 \text{ kg/cm}^2$$

(b) Also compute the equivalent radius of resisting section of this slab, given the radius of contact area of wheel load is 15cm.

sol $h = 15 \text{ cm}$

Radius of relative thickener,

$$r = \left[\frac{E h^3}{12 K (1 - \mu^2)} \right]^{1/4}$$

$$r = \left[\frac{21 \times 10^4 \times 15^3}{12 \times 3 (1 - 0.13^2)} \right]^{1/4}$$

$$r = 66.9 \text{ cm.}$$

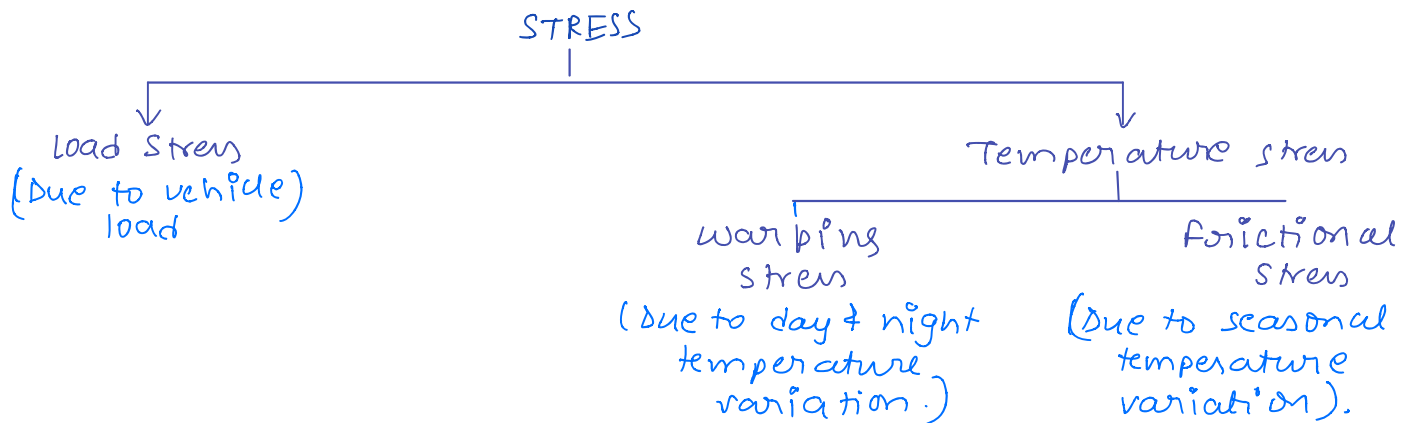
(b) $a = 15 \text{ cm}$

$$b \text{ (cm)} = \begin{cases} \sqrt{1.6a^2 + h^2} - 0.675h & a < 1.724h \text{ — True} \end{cases}$$

$$= \sqrt{1.6 \times 15^2 + 15^2} - 0.675 \times 15$$

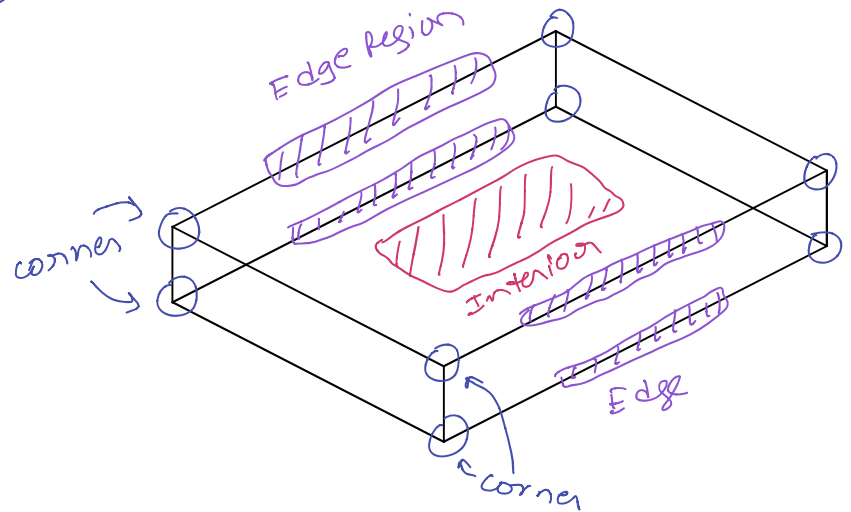
$$= 14.061 \text{ cm.}$$

ANALYSIS OF STRESSES :-



- critical location for development of stresses are as follows.

- (a) interior of slab
(b) edge of slab
(c) corner of slab
- } @ both top & bottom of slab.



① LOAD STRESS :-

- This stress develops in longitudinal direction of slab due to wheel load.
- It can be computed using westergaards theory as follows in kg/cm^2

(a) For interior region :-

$$(\text{kg/cm}^2) \leftarrow \sigma_{Li} = \frac{0.316 P}{h^2} \left[4 \log_{10} \left(\frac{l}{b} \right) + 1.069 \right]$$

(b) For edge region :-

$$\sigma_{Le} = \frac{0.572 P}{h^2} \left[4 \log_{10} \left(\frac{l}{b} \right) + 0.359 \right]$$

(c) For corner region :-

$$\sigma_{Lc} = \frac{3 P}{h^2} \left[1 - \left(\frac{a\sqrt{2}}{l} \right)^{0.6} \right]$$

P = wheel load (kg)

h = thickness of slab (cm)

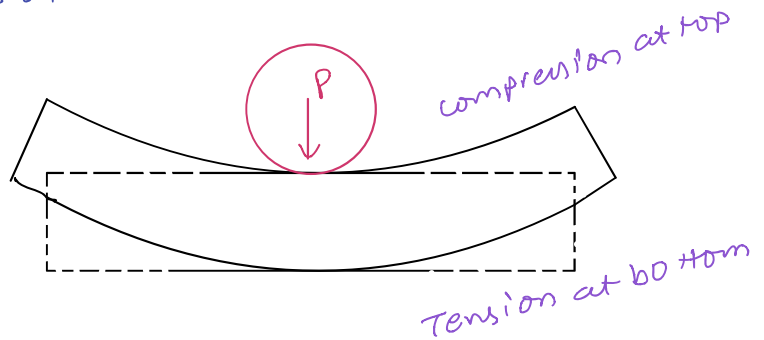
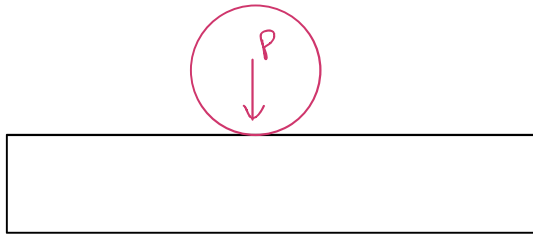
l = radius of relative stiffness (cm)

b = radius of equivalent resisting section (cm).

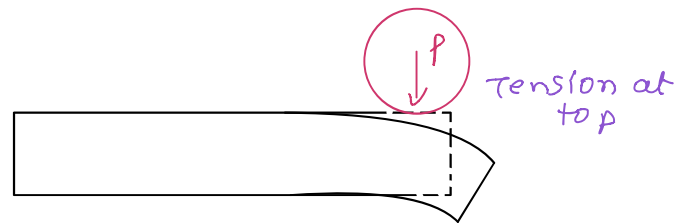
a = contact area radius.

- Nature of these stresses are as follows:-

(a) At interior/edge region.



(b) At corner region.

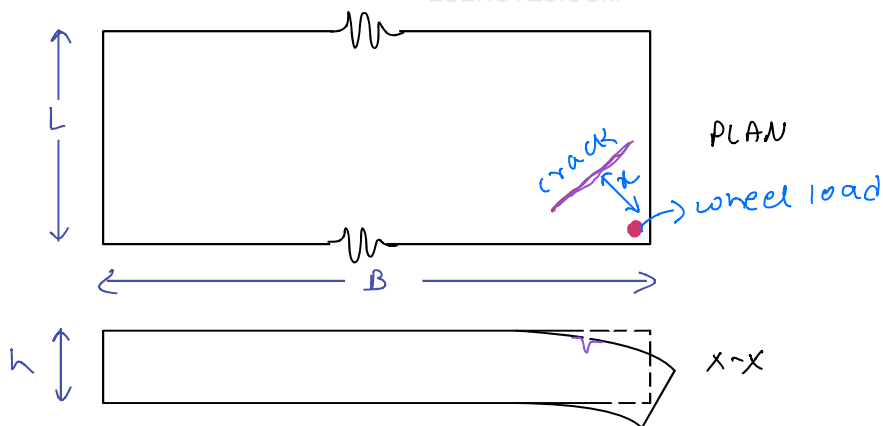


NOTE:- Position of crack due to corner loading bottom, is given by

$$x = 2.58 \sqrt{aL}$$

a = contact radius

L = radius of rel. stiffness.



② TEMPERATURE STRESS :-

These are developed in cement concrete pavement due to variation in the slab temperature and resistance against deformation by the weight of slab & friction b/w slab and ground surface.

- These stresses are caused by:-

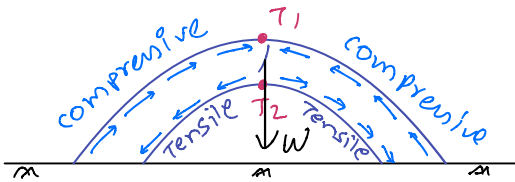
- Daily variation of temperature resulting in temp gradient across the thickness of slab.
- Seasonal variation resulting in overall change in

Slab temperature.

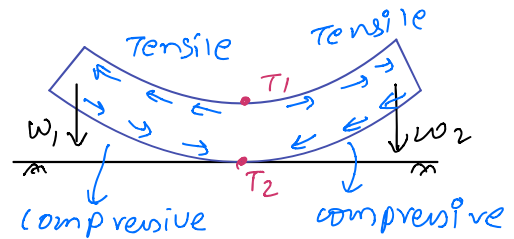
- These stresses are of two types.

(i) WARPING STRESS :- Due to daily variation of temperature leads to warping of slab due to temperature gradient across the thickness of the slab.

(Day time) ($T_1 > T_2$)



Night time ($T_2 > T_1$)



Day time

- During day time temp of slab/pavement is more at the top in comparison to temp. at bottom.
- Due to the slab tries to expand at top but weight of slab tries to resist it hence leads to development of compressive stress at top.
- Since temp at bottom of slab is less, it tries to contract, but weight of slab resists this contraction, leading to development of tensile stress at bottom.

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Night time

- During Night time temperature at top is less than bottom hence slab tries to contract at the top and expand at the bottom, which is resisted by weight of slab, that result in development of tensile stress at top and compressive stress at bottom of slab.

NOTE: (i) If the slab is assumed to be weightless then no warping stress would develop as there is no resistance to deformation due to temp. variation.

(ii) This stress comes out ^{to} be critical at the centre of the slab.

The magnitude of this stress are as follows :-

(a) At interior side :-
$$\sigma_{twi} = \frac{E \alpha t}{2} \left[\frac{c x + \mu c y}{1 - \mu^2} \right]$$

(b) At edge side :-

$$\sigma_{twe} = \max \left\{ \frac{c_x E \alpha t}{2}, \frac{c_y E \alpha t}{2} \right\}$$

E = mod. of Elasticity of conc. = $3 \times 10^5 \text{ kg/cm}^2$

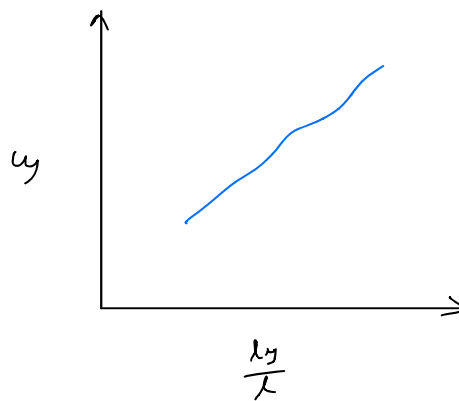
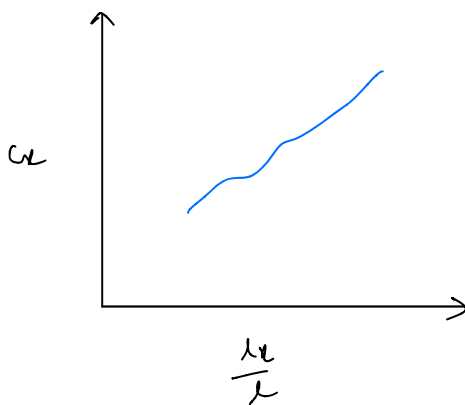
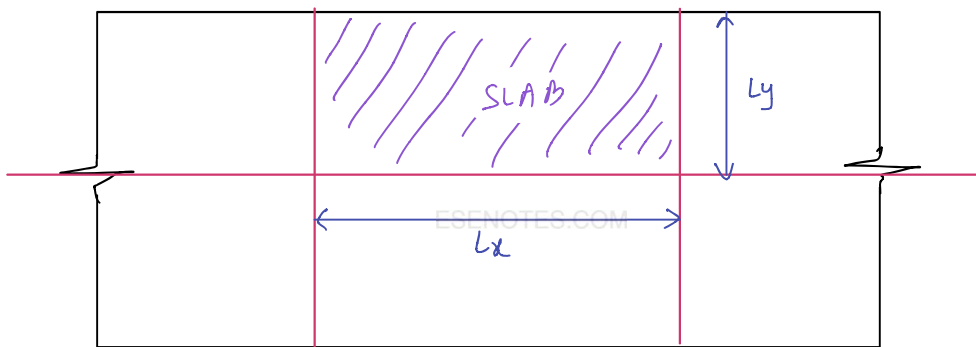
α = coeff of thermal expansion of concrete

T = Temp. diff b/w top and bottom of concrete slab

μ = poisons ratio = 0.15
 l = radius of area of contact.

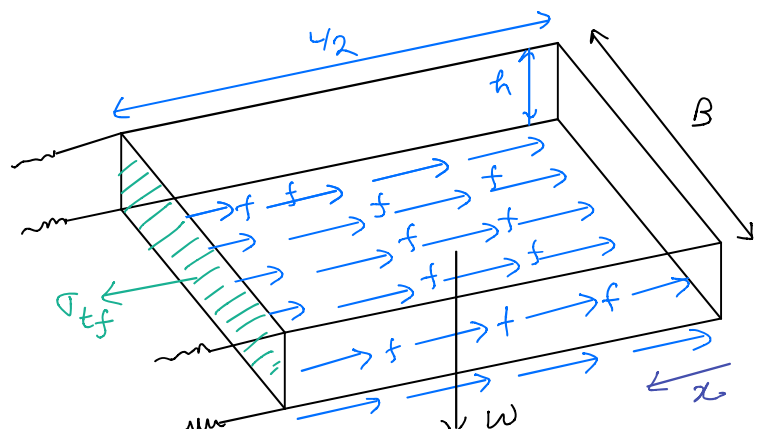
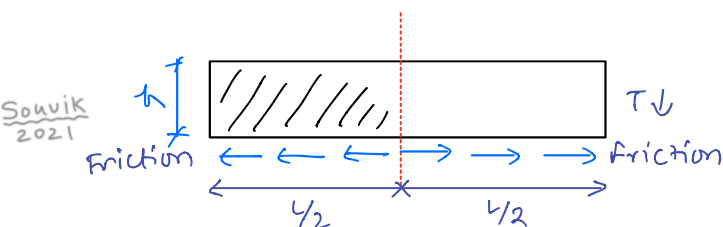
c_x, c_y are the factors calculated graphically. which depends upon $\frac{l_x}{l}, \frac{l_y}{l}$ respectively.

l_x = spacing b/w transverse joint of slab :
 l_y = " " longitudinal " " " "

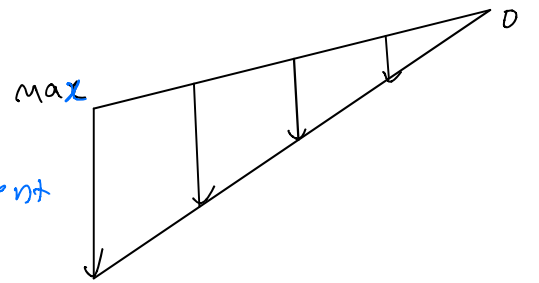


(ii) Frictional stress :-

WINTER :-



- These stresses are developed due to seasonal variation of temperature and in this case there is no temp. gradient across the thickness.



- During winter slab tries to contract due to temperature fall and this internal movement of slab is resisted by the friction which leads to the development of tensile stresses in it.

$$\sigma_{tf} (Bh) = F = \mu N = \mu W$$

$$\sigma_{tf} Bh = \mu r_c x (Bh x)$$

$$\sigma_{tf} = \mu r_c x$$

x = length of section from end.

NOTE :- $\sigma_{tf} \propto x$

Hence at $x=0$, $\sigma_{tf} = 0 \Rightarrow$ At corner it is zero.

at $x = l/2$, $\sigma_{tf} = \mu r_c l/2 = (\sigma_{tf})_{max} \Rightarrow$ At mid-section it is maximum

NOTE ② :- As slab is trying to contract or expand from both sides only half of slab is considered.

During summer due to rise in temp, slab tries to expand which is resisted by friction b/w slab and ground surface, that leads to the development of compr. stress in slab.

NOTE ③ :- Generally friction stresses are assumed to be constant along the length, but in actual it varies linearly.

Critical combination :-

(a) out of various wheel stress

(i) corner stress are maximum, due to discontinuity in both direction.

(ii) interior stress is minimum.

(iii) edge stress is in intermediate range.

(b) In combination of wheel load and temperature stress, edge region is most critical, hence designing is done using

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Region	Location	Load Stress	Warping stress		Friction		Worst combination of stresses
			Day	Night	Summer	Winter	
Interior	Top	C	C	T	C	T	Bottom, day time winter
	Bottom	(T)	(T)	C	C	(T)	$\sigma_i = \sigma_L + \sigma_{tw} + \sigma_{tf}$
Edge	Top	C	C	T	C	T	Bottom, day time winter
	Bottom	(T)	(T)	C	C	(T)	$\sigma_e = \sigma_L + \sigma_{tw} + \sigma_{tf}$
Corner	Top	(T)	C	(T)	C=0	T=0	Top, night time
	Bottom	C	T	C	C=0	T=0	$\sigma_c = \sigma_L + \sigma_{tw}$

- Q) A cc pavement slab of thickness 20cm is constructed over a granular sub-base having modulus of reaction 157 kg/cm². The max temp diff b/w the top and bottom of slab during summer day and night is 18°C. The spacing b/w the transverse contraction joint is 4.5m and that b/w longitudinal joint is 3.5m. The design wheel load is 5700 kg. Radius of contact area is 15cm, E for cc 3×10^5 kg/cm², $\mu = 0.15$, α for cc is $10^{-5} \text{ } ^\circ\text{C}^{-1}$ and friction $f = 1.5$. Use Westergaard's stress equation for wheel stresses and the given chart for warping stresses and find the worst combination of stresses at interior, edge and corner of slab.

$$\frac{l_x}{l} \text{ or } \frac{l_y}{l} : 2 \quad 4 \quad 6 \quad 8 \quad 10 \quad 12 \quad 14$$

$$C_x \text{ \& } C_y : 0.05 \quad 0.5 \quad 0.89 \quad 1.05 \quad 1.06 \quad 1.02 \quad 1.01$$

Sol (a) wheel load stress :-

$$\sigma_i = \frac{0.316 P}{h^2} \left[4 \log_{10} \left(\frac{l}{b} \right) + 1.069 \right]$$

$$l = \left[\frac{E h^3}{12 K (1 - \mu^2)} \right]^{1/4} = \left[\frac{3 \times 10^5 \times (20)^2}{12 \times 15 (1 - 0.15)} \right]^{1/4}$$

$$l = 60.77 \text{ cm.}$$

$$\frac{a}{h} = \frac{15}{20} = 0.75 (< 1.724)$$

$$b = \sqrt{1.6a^2 + h^2} - 0.675h = \sqrt{1.6 \times 15^2 + 20^2} - 0.675 \times 20$$

$$b = 14.06 \text{ cm}$$

$$s_{Li} = \frac{0.316 \times 5700}{(20)^2} \left[4 \log_{10} \left(\frac{60.77}{14.06} \right) + 1.069 \right]$$

$$\sigma_{Li} \leftarrow s_{Li} = 14.55 \text{ kg/cm}^2$$

$$s_{Le} = \frac{0.572P}{h^2} \left[4 \log_{10} \left(\frac{1}{b} \right) + 0.359 \right] = 21.15 \text{ kg/cm}^2$$

$$s_{Lc} = \frac{3P}{h^2} \left[1 - \left(\frac{a\sqrt{2}}{l} \right)^{0.6} \right] = 17.908 \text{ kg/cm}^2$$

(b) Warping stress

$$s_{twi} = \frac{E \alpha t}{2} \left[\frac{c_x + \mu c_y}{1 - \mu^2} \right]$$

$$\frac{l_x}{l} = \frac{450}{60.77} = 7.4, \quad \frac{l_y}{l} = \frac{350}{60.77} = 5.76.$$

From table

$$c_x = 1.002, \quad c_y = 0.843$$

$$s_{twi} = \frac{3 \times 10^5 \times 10 \times 10^{-6} \times 18}{2} \left[\frac{1.002 + 0.15 \times 0.843}{1 - 0.15^2} \right]$$

$$s_{twi} = 31.17 \text{ kg/cm}^2$$

$$s_{twe} = \frac{c_x E \alpha t}{2} = \frac{1.002 \times 3 \times 10^5 \times 10 \times 10^{-6} \times 18}{2}$$

$$s_{twe} = 27 \text{ kg/cm}^2.$$

$$s_{twc} = \frac{E \alpha t}{3(1-\mu)} \sqrt{\frac{a}{l}} = 10.52 \text{ kg/cm}^2$$

(c) frictional stress.

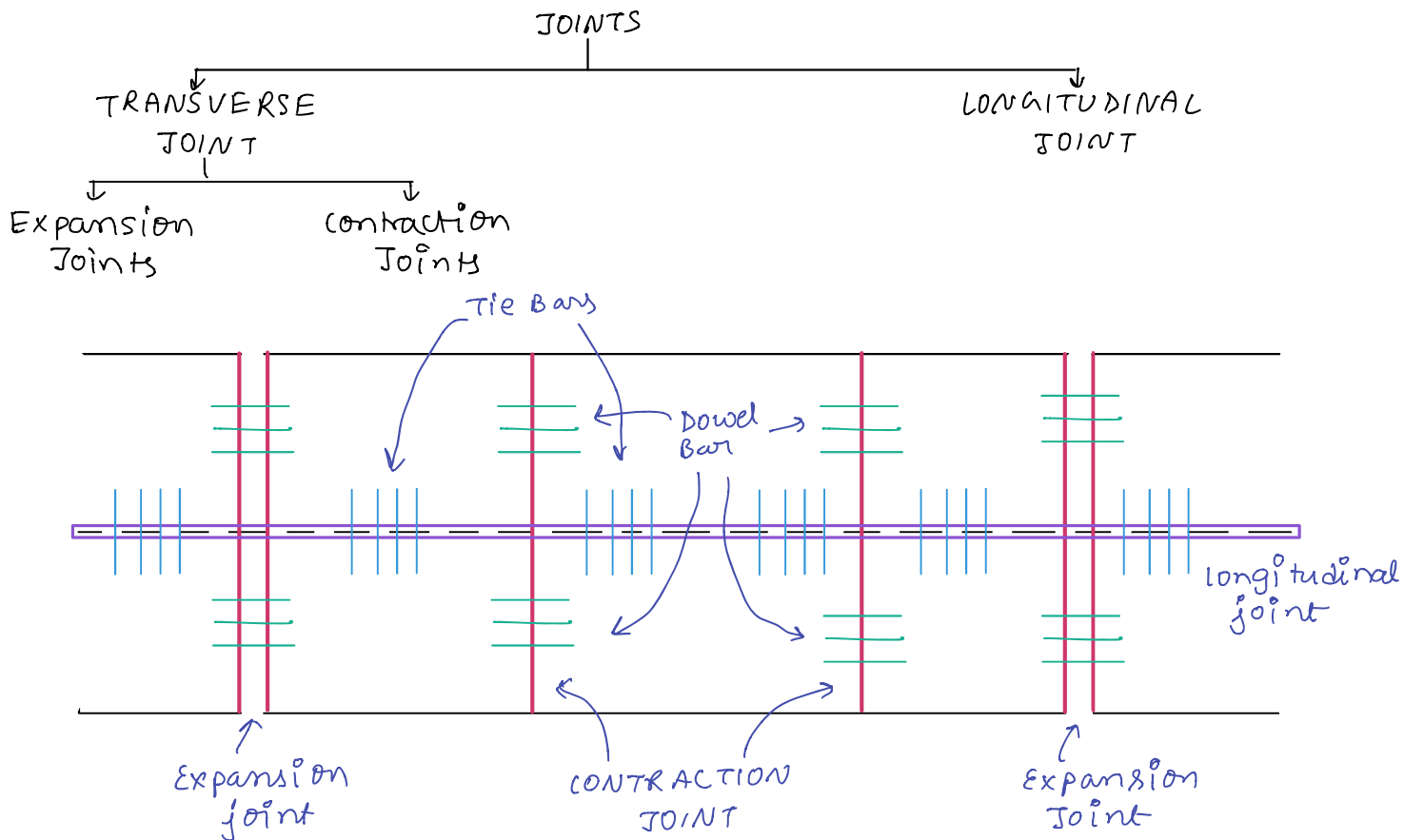
$$s_{t f i/e} = \frac{1}{2} f r L = \frac{1}{2} \times 1.5 \times 2400 \times 4.5 \times 10^{-4} \\ = 0.81 \text{ kg/cm}^2$$

$$s_i = \text{Bottom, winter, day} \\ = 14.55 + 31.17 + 0.81 = 46.2 \text{ kg/cm}^2.$$

$$s_e = 21.15 + 27 + 0.81 = 48.96 \text{ kg/cm}^2$$

$$s_c = 17.9 + 10.52 = 28.43 \text{ kg/cm}^2$$

ANALYSIS OF THE JOINTS :-



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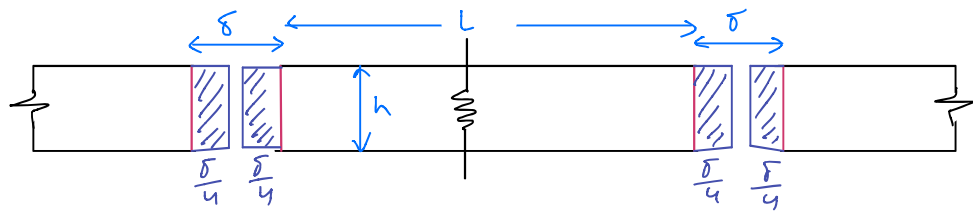
① EXPANSION JOINT

- The purpose of this joint is to allow expansion of pavement with rise in temperature w.r.t construction temp.
- To design this joint we find out the joint spacing for a given joint thickness of 2.5cm max as specified by IRC

NOTE:- AS per IRC thickness / width (δ) $\neq$ 2.5cm
spacing (L) $\neq$ 140m

- At the expansion joint dowels are provided which develops bending, bearing and shearing stresses and helps in load transfer.
- The fillers provided at the expansion joint are assumed to be compressed by 50% of its thickness during expansion. Hence the gap of joint should be twice the expansion in the pavement.

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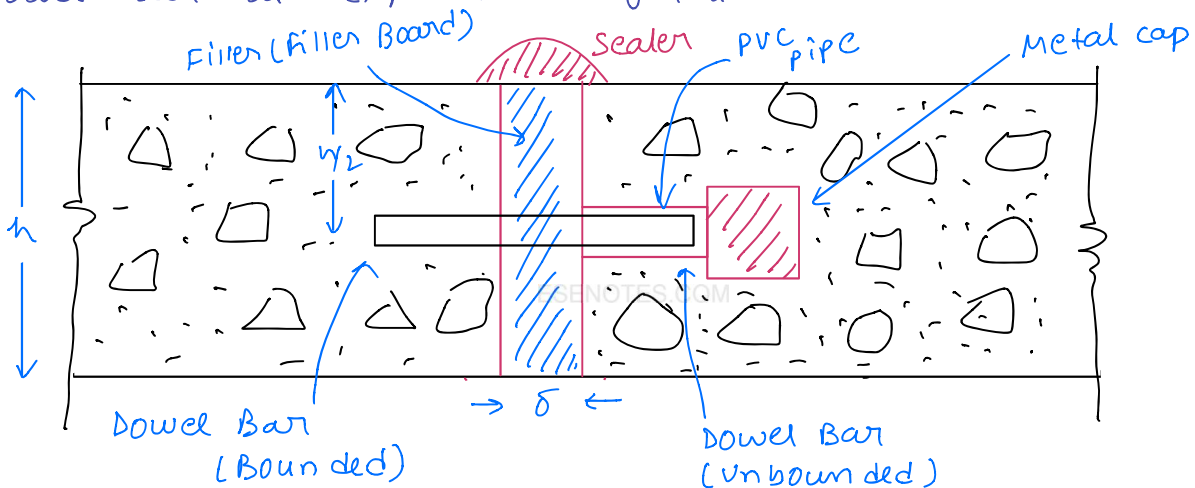
$$\text{Allowable expansion} = \frac{\delta}{4} + \frac{\delta}{4} = \frac{\delta}{2}$$

$$\frac{\delta}{2} = L \alpha \Delta T$$

NOTE, ΔT = Rise in temp. w.r.t construction temperature.
 $(\Delta T = T - T_c)$

$$L = \frac{\delta}{2 \alpha \Delta T}$$

- Dowel bar at expansion joint.



- Dowels are provided to serve the following purpose.

(a) To transfer the load from one slab to another

(b) To avoid differential settlement

As per IRC :-

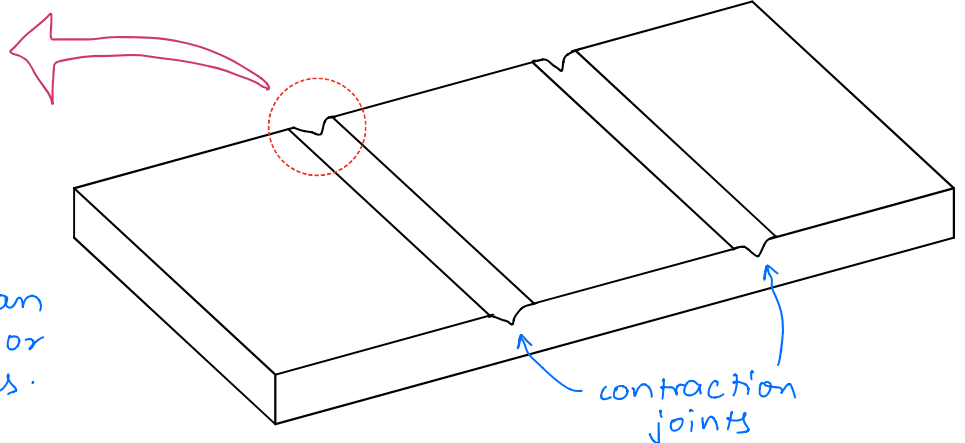
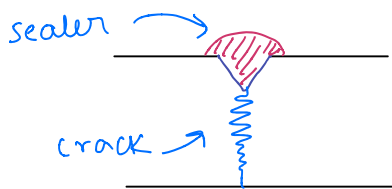
(i) $\phi = 25 \text{ mm}$
 (ii) length = 50 cm @ $\left\{ \begin{array}{l} 20 \text{ cm is bounded} \\ 30 \text{ cm is unbounded.} \end{array} \right.$

(iii) They are positioned at centre of slab (across thickness).

(iv) c/c spacing @ 300 mm or less.

② CONTRACTION JOINT :-

- It is provided to control the cracking of slab due to shrinkage and moisture variation.
- To regulate the crack i.e. to ensure that crack form at predetermined location, slab is weakened at certain intervals.
- These locations are contraction joints.



- contraction joints can be provided with or without dowel bars.

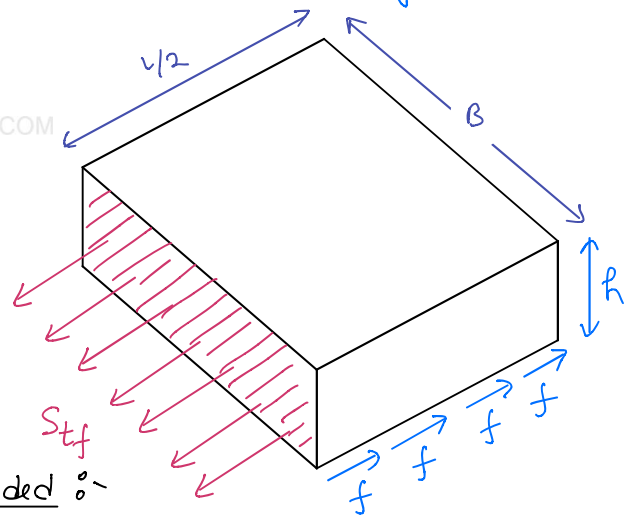
case (i) without dowel bars :-

$$S_{tf} (Bh) = F = fN$$

$$S_{tf} (Bh) = f \gamma_c \left(\frac{L}{2} \cdot B \cdot h \right)$$

$$L = \frac{2 S_{tf}}{f \gamma}$$

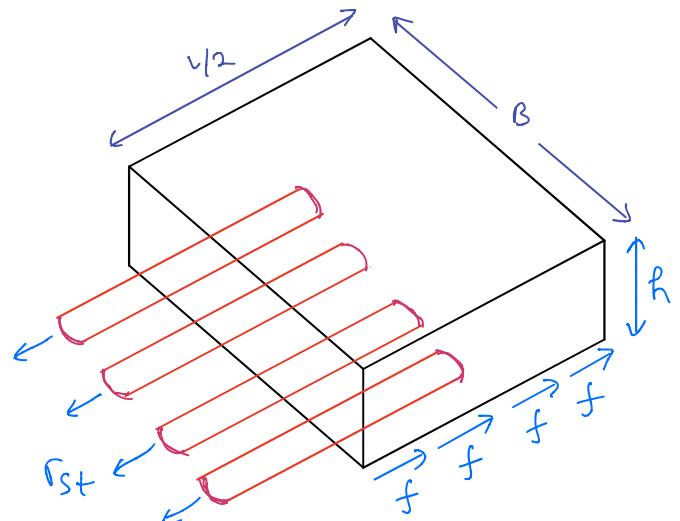
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case (ii) when dowel bar is provided :-

$$\sigma_{st} (A_{st}) = f \gamma_c \left(\frac{1}{2} B h \right)$$

$$L = \frac{2 \sigma_{st} A_{st}}{f L B h}$$



- As per IRC spacing of contraction joint

$$L \geq 4.5 \text{ m}$$

(ii) LONGITUDINAL JOINT :-

- These are provided along the length of pavement. It reduces warping stress.
- The normal width of slab is generally 3-5 m. If width of slab becomes more two slabs are provided along with longitudinal joint.
- Tie bars are also provided at longitudinal joint.
- These bars are not designed as load transfer device but ensures that the two slabs remain firmly together.
- These are bounded with concrete and we mostly use deformed bars in this case.
- As per IRC:-

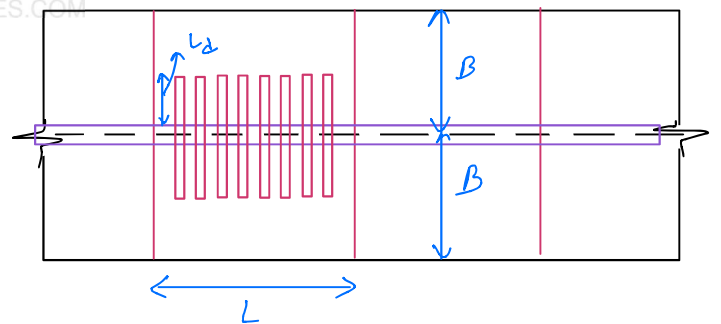
$\phi \neq 20 \text{ mm}$
Spacing $\neq 75 \text{ cm}$.

$$\sigma_{st} A_{st} = f_w = f \left(r_c \times l \times h \times \frac{b}{2} \right)$$

↓
per metre length

Area of steel for 1 m slab length

$$A_{st} = \frac{f_r c_b h}{\sigma_{st}} \quad \text{per metre length}$$



Let dia of tie bar be ϕ

$$\text{Area of one tie bar} = \frac{\pi}{4} \phi^2 = A'_{st}$$

$$\text{No. of tie bar in 1m length, } n = \frac{A_{st}}{A'_{st}} = \frac{A_{st}}{\frac{\pi \phi^2}{4}}$$

$$\text{Spacing b/w tie bars} = \frac{1000}{n} = \frac{1000}{\frac{A_{st}}{\frac{\pi}{4} \phi^2}}$$

$$\text{Development length, } L_D = \frac{\phi \sigma_{st}}{4 \tau_{bd}}$$

length of tie bar in slab,

$$L_T = 2L_D = \frac{\phi \sigma_{st}}{2 \tau_{bd}}$$

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Difference b/w Dowel & Tie Bar.

DOWEL BARS

- placed across transverse joint at the mid depth of the slab
- They transfer load from slab to another slab without preventing the joints from opening
- These are commonly made of round, smooth, epoxy coated steel bars.
- These bars reduces joint faulting and corner cracking

TIE BARS

- placed across longitudinal joint at the mid depth of the slab.
- They prevent lanes from separation and differential deflection.
- These are made of deformed epoxy coated steel.
- These bar reduces transverse cracking.

8) calculate the spacing b/w contraction joint by using following data.

- width of slab. = 4.52 m
- thickness of slab. = 25 cm
- coeff. of friction. = 1.5
- Allowable stress in steel. = 1400 kg/cm²
- Dia of bar = 12 mm
- spacing b/w the bar. = 300 mm

Sol =

$$\sigma_{st} (A_{st}) = f \left(r_c \frac{L}{2} B h \right) \quad n = \frac{\text{width}}{\text{spacing}} = \frac{4.52 \times 1000}{300} = 15$$
$$L = \frac{2 \sigma_{st} A_{st}}{f r_c B h} = \frac{2 \times 1400 \times 15 \times \frac{\pi}{4} \times 1.2^2}{1.5 \times 2400 \times 10^{-6} \times 452 \times 25}$$
$$L = 1172.86 \text{ cm.} = 11.72 \text{ m}$$

9) If width of expansion joint is 2cm & temp. at the time of construction of slab is 20°C, then find the spacing b/w expansion joint for maximum temperature of 45°C ($\alpha = 12 \times 10^{-6} \text{ } ^\circ\text{C}^{-1}$).

Sol =

$$\Delta T = 25^\circ\text{C}$$

$$\frac{\delta}{2} = L \alpha \Delta T$$

$$L = \frac{\delta}{2 \alpha \Delta T} = \frac{2}{2 \times 12 \times 10^{-6} \times 25} = 3333 \text{ cm}$$
$$L = 33.33 \text{ m.}$$

Highway materials and construction

- Materials used for pavement construction are as follows.

(i) SOIL-SUBGRADE :- It is formed via weathering of rocks. properties of which depends on mode of weathering.

- Soil subgrade must possess following properties.

- (a) stability
- (b) incompressibility
- (c) strength
- (d) good drainage
- (e) low permeability.

- To evaluate the strength following test are performed

(i) shear strength test :- direct shear, triaxial, vane etc.

(ii) BEARING TEST :- It's loading test carried out on subgrade soil in-site with the load bearing area or in lab also. which is used to ascertain overall stability of soil

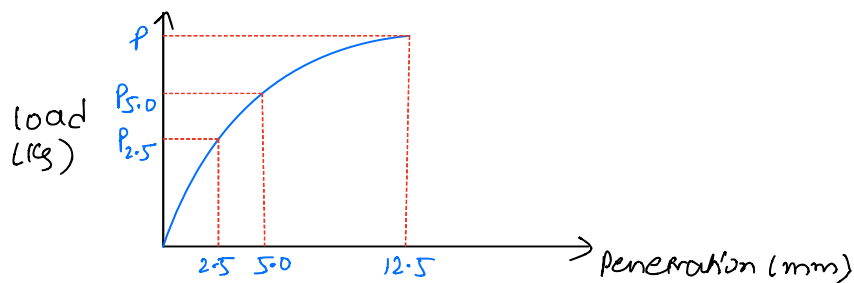
- Following test falls in this category.

(a) CBR Test :- It is an empirical test used to evaluate the soil subgrade.

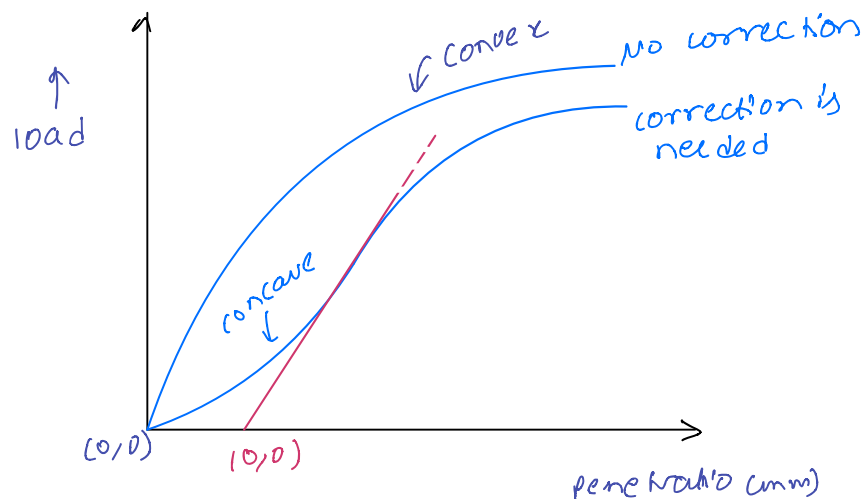
- In this test standard piston of dia 50mm is penetrated at the 1.25mm/min in the soil specimen (soaked in water for 4 days) placed in the mould of dia 150mm and its resistance against penetration is noted.

- The penetration is continued upto 12.5mm and the result is expressed in the form of curve showing resistance required for a particular depth of penetration.

- Resistance corresponding to 2.5mm and 5mm penetration is noted from the curve.



- This resistance (load) required for 2.5mm and 5mm penetration in soil is further compared with that of crushed stone standard sample.
- The load values for 2.5mm and 5mm penetration in standard sample is 1370 kg (70 kg/cm²) and 2055 kg (105 kg/cm²)
- CBR is then defined as ^{ratio of} load carried by specimen to the standard sample.
- Generally with increase in depth CBR value decreases.
- But if in some case $CBR_5 > CBR_{2.5}$, the test is repeated and whatever result is obtained is adopted.
- If the specimen has surface irregularity, the initial portion of curve may have concavity upwards, which can be eliminated by drawing the tangent at the point of inflection and shifting the origin to the intersection of tangent and x-axis.

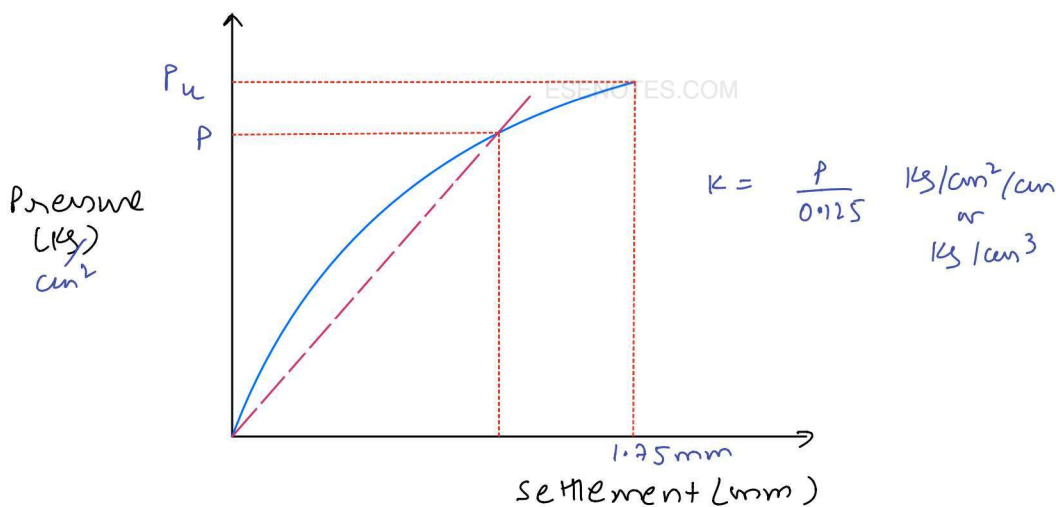


Mar 2 (b) PLATE BEARING TEST

- This test is used to evaluate the support capability of subgrade, bases & in some cases complete pavement.
- Data from this test is applicable for design of both flexible and rigid pavement.
- In this test, compressive stress is applied to the soil or pavement layer through rigid plate relatively larger in size and the deflections are

measured for different stress value.

- In this test site is prepared by removing the loose material from top and 75cm dia plate is placed in contact with soil subgrade.
- A seating equivalent load of 320 kg (giving the pressure of 0.07 kg/cm^2) is applied over it and released after few second.
- The settlement of dial gauge is now set to zero.
- Now load over the plate is applied and corresponding settlement is noted over three dial gauges (avg of 3 reading is considered as result) upto an extent total settlement of 1.25 mm is achieved.
- Result is then plotted in the form of curve and load / pressure corresponding to 1.25 mm or 0.125 mm is noted from curve and is known as "modulus of subgrade k_{75} ".



- If 30 cm dia plate is used then as per IRC.

$$k_{75\text{cm}} = 0.5 k_{30\text{cm}}$$

But theoretically, $k_{75} = 0.4 k_{30}$

$$\left[\text{as } \Delta = \frac{1.18 \text{ Pa}}{E} \right]$$

(II) AGGREGATE :-

- This are obtained from weathering of rocks and mineral materials eg; sand, gravel.
- They are used for preparation of bituminous or cement concrete.

- By volume aggregates generally accounts for (92-96)% of bituminous concrete and about (70-80)% of cement concrete.
- For these aggregates to be used as pavement material, following properties are required.
 - (a) STRENGTH :- Resistance against crushing or gradual loading.
 - (b) HARDNESS :- Resistance against abrasion.
 - (c) TOUGHNESS :- Resistance against impact loading.
 - (d) SHAPE OF AGGREGATE :- Those shape of aggregates are required which gives either higher strength or higher workability.
 - (e) ADHESION WITH BITUMEN :- The aggregate used in bituminous pavement should have less affinity for water when compared with bituminous material, otherwise the coating would be stripped off the aggregate in the presence of water.
 - (f) DURABILITY :- They should be highly resistant against temp, humidity, rain, snow etc.
 - (g) INERTNESS :- Aggregate should be inert in nature i.e. chemically inactive else they would react with binding material or bituminous material.

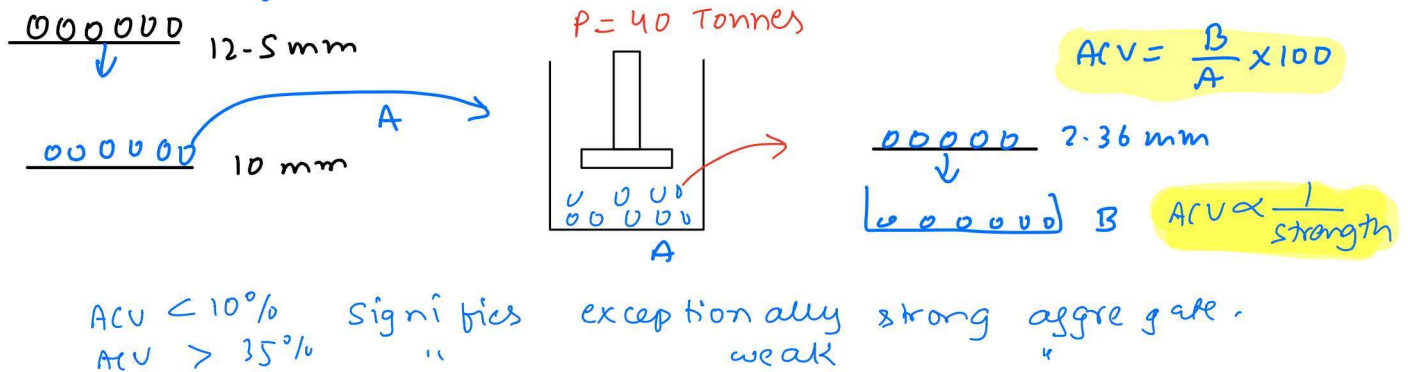
- Testing of aggregates can be done by any of the following methods.

(i) CRUSHING TEST :- (Also refer from BMC)

- Strength of aggregates directly governs the strength of the concrete, in which it is used for construction.
- It is defined as the ability of the aggregates to resist gradual loading.
- It is determined in terms of parameter "aggregate crushing value" which is found using Aggregate crushing value test.
- In this test sample of aggregate passing thru 12.5mm

sieve and retained over 10mm sieve is subjected to gradual loading with the help of 40 tonne plunger for 10 mins.

- Sample is then passed thru 2.36mm sieve and wt. of particles passing through the sieve is noted, which is then expressed in terms of original weight of aggregate is termed as "Aggregate crushing value" (ACV)
- For aggregates to be used for pavement construction
ACV $\nless 30\%$ $\rightarrow$ surface course.
for general construction, ACV $\nless 45\%$ $\rightarrow$ Base course



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(ii) ABRASION TEST (Hardness of Aggregate)

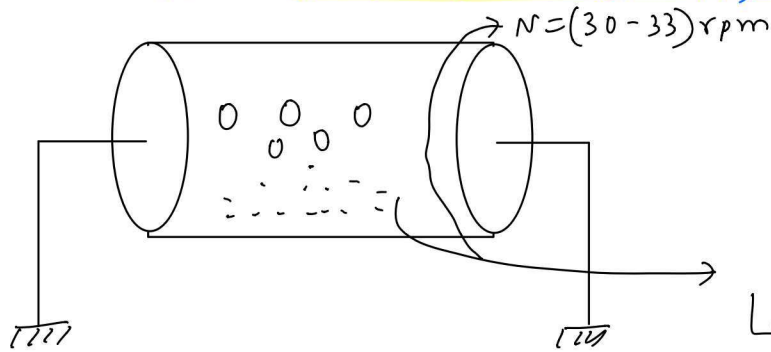
- It directly governs the hardness of concrete in which it is used for construction
- It is defined as resistance against wear and tear or abrasion.
- It is referred in terms of parameter termed as "AGGREGATE ABRASION VALUE" and is found using following test

- (i) DEVAL Abrasion test
 - (ii) DOWRY Abrasion test
 - (iii) LOS-ANGELES " " $\rightarrow$ code suggest
- process is same but parameters are diff

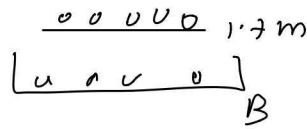
- In this test sample of the aggregate is subjected to wear and tear in rotating cylinder having "Steel balls" in it.

The sample is then passed thru, 1.7mm sieve and wt of agg. passing through the sieve is noted which when expressed in terms of original wt of agg. is termed as "AGGREGATE ABRASION VALUE"

- For pavement construction, $AAV \geq 30\%$ → Surface course
- For construction work, $AAV \geq 50\%$ → Base course



STEEL BALL } Los-angeles test
 $\phi = 48 \text{ mm}$
 $w = (390 - 450) \text{ gm}$



$$AAV = \frac{B}{A} \times 100$$

$$(AAV \propto \frac{1}{\text{Hardness}})$$

For WBM → $AAV \geq 40\%$

NOTE:- here a parameter can also be used to represent the hardness of aggregate termed as coefficient of hardness = $\frac{20 - \text{loss in wt (gm)}}{3}$

$$\text{loss in wt} = A - B$$

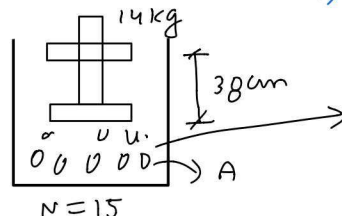
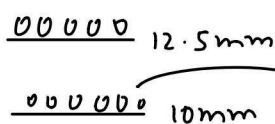
(iii) IMPACT TEST :-

- Toughness of aggregate directly governs the toughness of concrete in which it is used for construction.
- It is defined as the ability of agg to resist impact loading
- It is represented in terms of the parameter "AGGREGATE IMPACT VALUE"
- It is determined by performing "Aggregate impact value test"
- In this test sample of the agg passing thru. 12.5 mm sieve and retained over 10 mm sieve is subjected to impact loading with the help of 14 kg hammer that is allowed to fall freely from height of 38 cm over the sample 15 times.

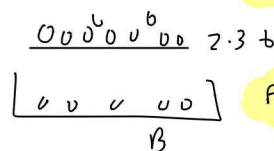
- Sample is then passed thru 2.36 mm sieve and the weight of the agg passing thru the sieve is noted and is expressed in terms of the original weight of aggregate is termed as "Aggregate impact value" (AIV)

- For pavement construction, $AIV \geq 30\%$
- For general construction, $AIV \geq 45\%$

→ Surface course, $AIV \geq 35\%$
 For Base course



$$AIV = \frac{B}{A} \times 100$$



$$AIV \propto \frac{1}{\text{Toughness}}$$

For bituminous macadam $\rightarrow$ A.V. $\neq$ 40%.

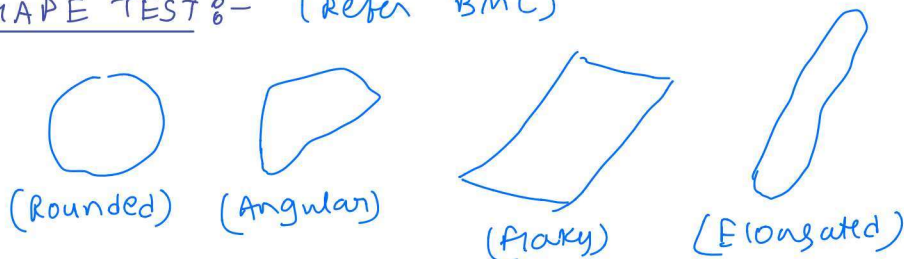
3 Mar (iv) SOUNDNESS TEST :-

- This test is performed to check the durability and resistance against weathering action of aggregates. (SIK).
- Aggregates of specified size are subjected to cycles of alternate wetting in saturated solution of sodium sulphate or $MgSO_4$ for (16-18) hrs and then dried in an oven at the temp. of (105-110°C) to a constant weight for 3-4 hrs. (Generally such 5 cycles are repeated).
- After 5 cycles the loss in the wt of aggregates is determined by sieving out all the undersized particles.
- Loss in weight should not exceed by 12% when tested with sodium sulphate and 18% when tested with magnesium sulphate.

NOTE:- $MgSO_4$ solution is comparatively more strong than Na_2SO_4 .

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(v) SHAPE TEST :- (Refer BML)



Shape of aggregates governs both workability and strength of the mixture

- Rounded aggregates leads to the formation of workable mix having low strength. As lesser is the area to be lubricated in this case & poorer is the interparticle locking and bond strength.
more workable, negligible strength.
- Angular aggregates leads to the formation of strong mix having less workability as better is the interparticle locking and bond strength but more is the area to be lubricated in this case.
less workable, but more bond strength, strength $\uparrow$
- Flaky and elongated aggregates leads to the formation of poor mix. which neither posses strength nor workability. as poor is the interparticle locking and higher is the area to be lubricated in this

car.

Neither strength nor workable
 Pen → elongated agg. pag → flaky agg.

NOTE: - Excellent concrete is made by using crushed stone but the particles should be roughly cubical in shape.

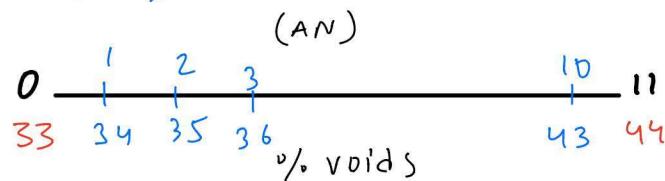
Measurement of shape

- Angularity or roundness of aggregates is measured in terms of parameter "ANGULARITY NUMBER" (AN)

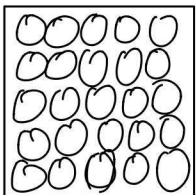
which represents the volume of voids in the sample

Angularity ↑, roundness ↓ and vice versa.

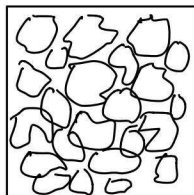
- (AN) varies in the range of (0-11), if % voids in the sample is 33%, AN = 0
 % void is 44%, AN = 11



$$\text{Angularity No} = \% \text{void} - 33$$



less voids
 (Round)



more voids
 (Angular)

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Q) The weight of coarse agg having $G = 2.65$, which is completely filling into a cylinder of volume 0.003 m^3 is 5247 gm . What is its angularity number and comment upon the shape of aggregates.

sol

$$W = 5247 \text{ gm}$$

$$G = 2.65$$

$$G = \frac{r_s}{r_w}$$

$$r_s = G r_w = 2.65 \times 1 \frac{\text{gm}}{\text{cm}^3} = 2.65 \times \frac{1}{10^6} = 2.65 \times 10^{-6} \text{ g/m}^3$$

$$r_s = 2.65 \times 10^{-6} \text{ g/m}^3$$

$$\frac{W_s}{V_s} = 2.65 \times 10^{-6}$$

$$V_s = \frac{5247}{2.65} \times 10^{-6} = 1.98 \times 10^{-3} \text{ m}^3$$

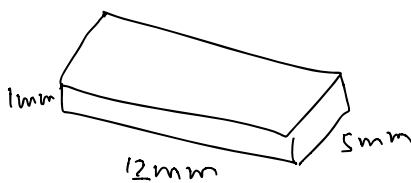
$$\text{Total vol} = 0.003 \text{ m}^3, \quad V_v = V - V_s$$

$$\% \text{Void} = \frac{V_v}{V} \times 100 = \frac{0.003 - (1.98 \times 10^{-3})}{0.003} = 34 \%$$

∴ Angularity No - ①, Rounded

FLAKY

- Flakiness of the aggregate is measure in terms of parameter "FLAKINESS INDEX"
- Flaky particles are those least lateral dimension of which is smaller than $0.6 \times \frac{3}{5}$ of its mean dimension
- Flakiness index is defined as % of flaky particle in the sample and is determined using flakiness index test.
- This test is not applicable for the particles having size smaller than 6.3mm
- In order to perform this test sufficient quantity of agg must be considered, such that 200 pcs of each fraction can be gauged.
- Particles of each fraction are passed turn by turn through the respective opening over the thickness gauge and weight of the aggregates passing through this opening noted and when expressed in terms of original weight of aggregates is termed as flakiness index.
- Flaky aggregates must not be more than 15% in general to be used for preparation of concrete



$$\bar{x} = \frac{1+12+5}{3} = 6\text{mm}$$

$$y = \frac{3}{5} \bar{x} = 3.6\text{mm}$$

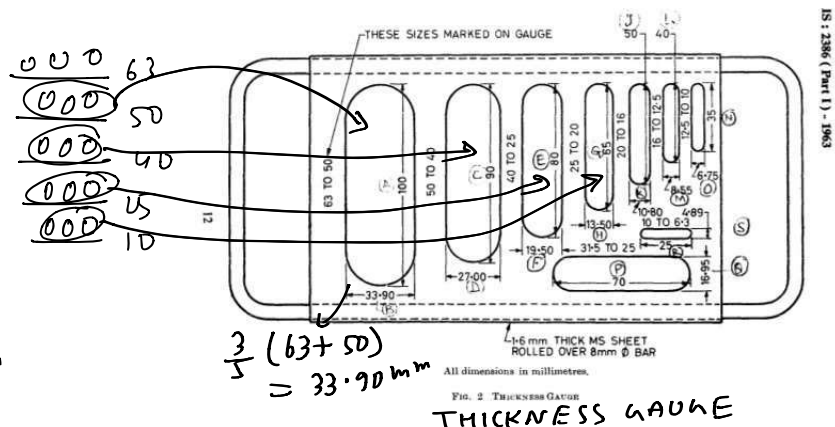
least dim (1mm) < 3.6mm

↓
Flaky agg.

if agg passes it is flaky

As per IRC:-

FI	≥ 25%	for bituminous concrete
FI	≥ 15%	for WBM



Elongated

- Elongated agg are those, greatest size of which is greater than $1.8 \times (\text{Mean size})$.
- Elongation of the agg. is measured in terms of the parameter elongation Index. which represents the % of elongated particles in the sample
- This test is not applicable for agg. having size smaller 6.3mm
- In order to perform this test suff quantity of agg. must be taken such that 200 pieces of each fraction can be gauged.
- Particles are then passed thru respective opening over the length gauge and the agg. retained over this openings is weighed and expressed in terms of original wt. of agg and referred as elongation index.

from prev exampk

$$\bar{x} = 6 \text{ mm}$$

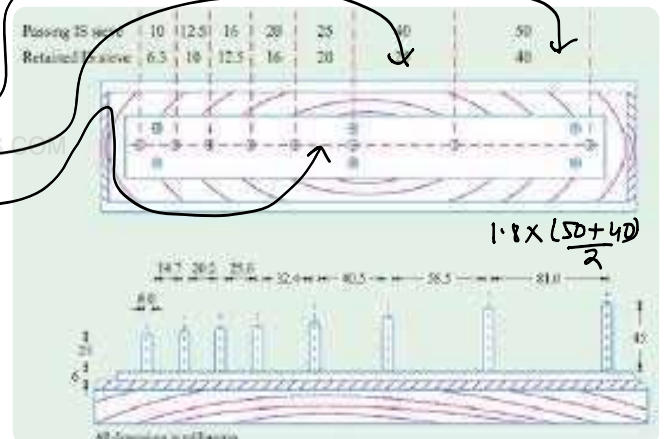
$$y = 1.8 \times 6 = 10.8 \text{ mm}$$

Now, 12 mm $> 10.8 \text{ mm}$

↓
longest
dim.

flaky as well as elongated.

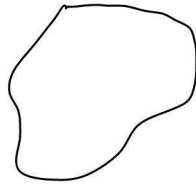
50	50
00	40
00	15
00	00
00	10



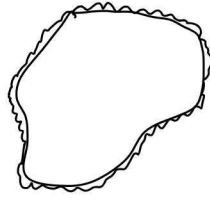
ELONGATION GAUGE

TEXTURE OF AGGREGATE :-

- Surface texture of aggregate represent relative % of area of aggregates i.e smooth or rough
- Smooth textured aggregate leads to the formation of workable concrete but having low strength as lesser is the area to be lubricated in this case and lower is the bond strength and inter particle locking.
- Rough aggregates leads to the formation of strong concrete having low workability as more is the area to be lubricated and more the inter particle locking and bond strength



SMOOTH



ROUGH

- workable high • less workable
- less bond strength • high bond strength

• for workable $\rightarrow$ SMOOTH + ROUNDED
 for strength $\rightarrow$ ROUGH + ANGULAR

(vi) SPECIFIC GRAVITY TEST :-

- It is defined as ratio of mass of solid to that of an equal volume of standard fluid (water) at a specified temperature.
- As the aggregates may contain water permeable voids, so to measure the specific gravity of aggregates two terms are used.

$\rightarrow$ (V_s) $\rightarrow$ of solids

(a) apparent sp. gravity (G_{app}) :- It is computed on the basis of net volume of agg. i.e. volume excluding water permeable voids

$$G_{app} = \frac{M_o}{\frac{V_n}{\rho_{water}}}$$

V_n = net volume of aggregates excluding voids.

(b) Bulk specific gravity (G_B) :- It is computed on the basis of total vol. of aggregates including water permeable voids.

$$G_B = \frac{M_o/v}{\rho_w}$$

v = vol. of agg. including voids.

- for aggregates to be used in pavement construction

sp. gravity = (2.6 - 2.9)

(vi) WATER ABSORPTION TEST :-

- The difference of apparent and bulk sp. gravity signifies the amount of water permeable voids of aggregate.

- we can measure the volume of such voids by weighing the aggregates dry and in a saturated surface dry condition.

with all the permeable voids filled with water.

- water absorption must not be greater than 0.6% by weight of aggregates. (generally it is in the range of 0.1-2)%

(vii) BITUMEN ADHESION TEST :-

- Bitumen adheres well to all normal types of aggregates used for pavement construction.
- In the presence of water stripping of binder from the coated aggregates takes place.
- Several test are available to find the adhesion of bitumen binder to an aggregates in the presence of water.
- As per IRC, static immersion test is used, in which aggregates coated with bitumen is immersed in water at 40°C for 24hrs.
- As per IRC max stripping value $\neq$ 5%

(III) BITUMEN :-

- It is obtained fractional distillation of crude oil.
- It is used for preparation of riding surface, bituminous maintenance material
- physical properties of bitumen are as follows.

- Specific gravity $G = 1$.
- Black in color.
- Free carbon content is less.
- less temperature susceptibility.
- more resistant to water.
- soluble in CS_2 and CCl_4 .
- chemically inert.
- oxidises slowly.
- highly viscous.

- Grading of bitumen is done on the basis of its viscosity and is termed as VISCOSITY GRADIENT (VG).

- costly, used in high stress area.
eg $\rightarrow$ Toll plaza, truck terminal.
- VG-40 $\rightarrow$ (3200 poise)
 - VG-30 $\rightarrow$ (2400 poise)
 - VG-20 $\rightarrow$ (1600 poise)
 - VG-10 (800 - poise) $\rightarrow$ spraying.
- $\Rightarrow$ when used in India for paving applic
 $\Rightarrow$ used for paving in cold climate.

5 Mar (D) TAR :-

- It is manufactured by destructive distillation of coal or wood. Hence free carbon content is comparatively more.
- It is also more susceptible for temperature.
- Its resistance against water is comparatively less.
- It is soluble in "TOLUENE".
- Here the grading of the TAR is done on the basis of strength as follows :- RT = Road Tar.
Suitable to be used in
 - (a) RT-1 : Surface dressing in cold weather.
 - (b) RT-2 : Surface dressing in normal climate
 - (c) RT-3 : " & Renewal coat
 - (d) RT-4 : Premix Tar macadam.
 - (e) RT-5 : Grouting.

Strength : RT-5 > RT-4 > RT-3 > RT-2 > RT-1

(V) CUTBACK BITUMEN :-

- It is obtained by blending bitumen binder with volatile dilutant or solvent in order to reduce its viscosity to desired range.
- It is used in following conditions.
 - (a) For application in cold regions.
 - (b) For Tack coating without heating.
 - (c) For soil stabilization.
 - (d) For bitumen mixes.
- cutback bitumen is graded on the basis of its curing i.e. on the basis of the relative speed of evaporation as follows.
 - (i) RC (Rapid curing)
 - (ii) MC (medium curing)
 - (iii) SC (slow curing).

- The cutbacks are designated by numerals representing progressive thickness or viscosity
- For eg RC-2 is more thick than RC1, but RC2, MC2 and SC2 has same viscosity. The only diff b/w them is the rate of vapourisation.

(vi) BITUMEN EMULSION :-

- Emulsion is a two phase system consisting of immiscible liquid (two).
- The bitumen or TAR content in the emulsion ranges from (40-60)% and remaining portion is water.
- The avg size of particles in this emulsion is about 2mm.
- These are specially used in maintainance or patch repair work.
- It can be used even in wet weather condition.
- This emulsions are of two types :-

(a) ANIONIC BITUMINOUS Emulsion

(b) CATIONIC Bituminous Emulsion. (India, it is used).

- Grading of bituminous emulsion is done on the basis of speed at which water evaporates & bitumen particles separate from water as follows.

(i) Rapid setting emulsion. (RS)

(ii) Medium " " (MS)

(iii) Slow " " (SS)

NOTE!:- Here word "SETTING" does not mean loss of plasticity rather it means the time taken by the bitumen to separate from water.

(vii) ASPHALT :-

- Produced by heating the mixture of Bitumen, aggregate and sand into a composite mix.

TESTING OF BITUMEN :-

(i) VISCOSITY TEST :-

- It is the property of bitumen which resist flow due to internal friction.
- It is determined by "efflux viscometer" or "SAYBOLT FUROL".
- In this test bitumen is placed at constant temperature and made to pass through an orifice.
- The time required to pass is measured in seconds and is used to indicate the viscosity of the bitumen.
- Temperature during the test is in range of $(25-50)^{\circ}\text{C}$.
- The dia of orifice is $(3-10)$ mm and vol. of bitumen used is $(50-200)$ ml.
- Viscosity of the tar is determined by "tar viscometer".



(ii) DUCTILITY TEST :-

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- It is the ability of the bitumen to be stretched without breaking.
- It is measured by stretching a standard briquet of area 1cm^2 at temp of 27°C and rate of 5cm per min.

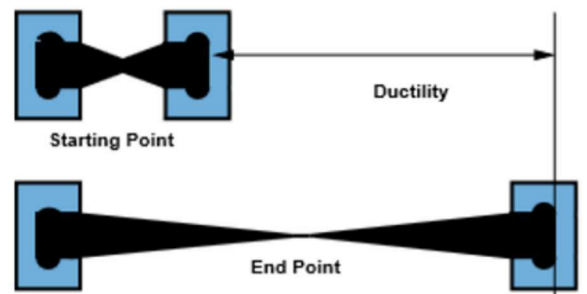
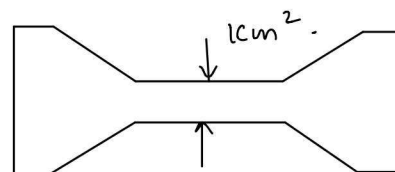


Figure 23.2: Ductility Test

- The distance in (cm) that the briquette can be stretched before breaking is related to its ductility.
- Its value varies from $(5-100)$.
- A min value of 50 is commonly specified.
- Min. ductility of grade 45 & above is 75cm. as per IRC.



(iii) PENETRATION TEST

- It is the measure of hardness or softness of bitumen, it measures the distance a standard blunt-pointed needle vertically penetrates a sample of material at 25°C .
- The load being of 100 gm and time of penetration is 5 sec.
- Unit of measurement is $\frac{1}{10}\text{ mm}$.
- Here result is reported as x/y for eg. a result of $80/100$ means penetration of $(8-10)\text{ mm}$
$$80 \times 0.1 = 8\text{ mm}$$
$$100 \times 0.1 = 10\text{ mm}$$
- TAR is not being tested by this method as it is comparatively very soft compared to bitumen.



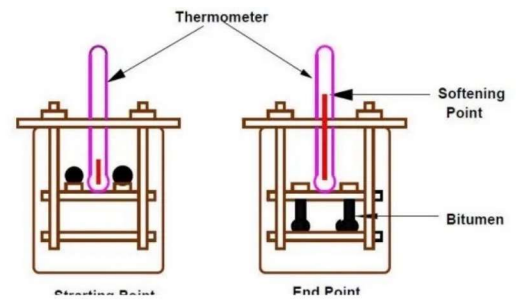
Mar 6 (iv) SOFTENING POINT TEST :-

- The softening point is measured by RING & BALL TEST
- Softening point is the temperature at which bitumen attains a particular degree of softness under standardized condition.
- The temperature at which a standard steel ball placed on the layer of bitumen kept in standard ring passes through bitumen layer and touches the bottom plate kept at a distance of 2.54 cm is the softening point.
- The softening point of bitumen generally varies b/w $(35-70)^{\circ}\text{C}$.
- It can also be understood that softening point is the temperature at which bitumen passes from solid to liquid state of consistency.

NOTE:- For satisfactory performance and avoidance of "Bleeding" bitumen should have a softening point $(5-10^{\circ}\text{C})$ above the max^{m} atmospheric temp. of the location.

(V) FLASH & FIRE POINT TEST :-

- At high temperature, bitumen becomes volatile and thus catches fire which is very dangerous.



RING & BALL APPARATUS.

- Thus it is necessary to find the temperature at which different grades of bitumen catches fire.
- The flash point is the lowest temperature in $^{\circ}\text{C}$ at which the application of a test flame causes the vapours from the bitumen to catch fire immediately in the form of flash.
- It is approximately 175°C
- The fire point is the lowest temperature at which the application of test flame causes the bitumen to ignite and burns for at least 5 seconds under specified test conditions.
- The safe limit for heating bitumen is generally 50°C below the flash point.
- It is measured using "PENSKEY MARTIN"

(VI) FLOAT TEST :-

- Normally the consistency of bituminous material can be measured by either penetration test or viscosity test.
- But for certain range of consistencies, these tests are not applicable & float test is used.
- The float test is a modified viscosity test.
- Here consistency is reported in terms of float value, which represents time and seconds required for a small plug of bitumen, which is held in an open mould attached to the bottom of float to become sufficiently fluid at temperature of approximately 50°C .

(VII) HEATING TEST :-

- When a bituminous material is heated it loses its volatile content and hardens.
- The less the loss of weight on heating better is the bitumen.
- The test is conducted by an accelerator test on 50 grams of bitumen placed in a container and the temperature is increased upto 160°C for 5 hours.
- Loss in weight of bitumen is then noted and it must not be more than 1% (49.5gm).

MIX DESIGN

- Mix design can be done by any of the following methods:
 - (i) Marshall mix design. $\rightarrow$ used in India. (most popular).
 - (ii) Hubbard - Field method.
 - (iii) Huveem - Method.
 - (iv) Smith trial axial.
- Mix design is carried out for the following objectives :-
 - (a) To have sufficient bitumen to ensure durable pavement.
 - (b) " " " strength to resist shear deformation.
 - (c) " " " air voids in compacted bitumen.
to allow additional compaction by traffic.
 - (d) To have sufficient workability.
 - (e) To have sufficient flexibility, to avoid cracking.
- constituents of mix design consists of following.
 - (a) coarse aggregate :- crush stones, granite. (provided to impart strength).
 - (b) fine aggregate :- sand and rock dust, provided to impart stiffness.
 - (c) Filler :- cement, lime, fills the voids of FA, provided to impart stiffness.
 - (d) Binder :- Bitumen, Asphalt, tar :- provided to impart adhesion.

- The design mix involves three important steps.

- (a) Selection of aggregates.
- (b) Aggregates gradation.
- (c) Proportioning of aggregates.

MARSHALL MIX DESIGN :-

- This mix design determines the optimum bitumen content.
- For evaluation of performance of bitumen flow test and stability test is performed.
- The stability portion of the test measures maximum load supported by the test specimen at a loading rate of 50.8 mm/min .
- Load is applied to the specimen till failure.
- Stability is then defined as maximum load carried by the specimen at standard temp. of 60°C .
- Approximately 1200 gm of aggregates and fillers is heated to the temperature of $(175-190)^\circ\text{C}$.
- Bitumen is heated to $(120-125)^\circ\text{C}$ with first trial % of bitumen in range of $(3.5-4)\%$ by wt.
- The heated aggregates & bitumen are mixed at temperature of $155-160^\circ\text{C}$.
- The mix is then placed in pre-heated mould at $(135-150)^\circ\text{C}$.
- The thickness of the specimen is 63.5 mm and its diameter is 101.6 mm .
- Vary the bituminous content in the next trial by 0.5% & repeat the process upto the 8% bituminous content.

March 8 # PROPERTIES OF MIX :-

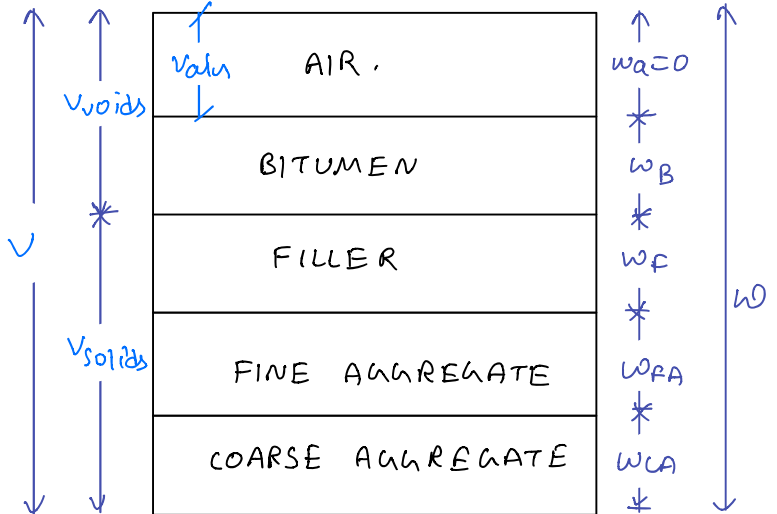
(i) Theoretical specific gravity :- (G_t)

- It is the sp. gravity of the mix without considering air voids.

$$G_t = \frac{W_{total}}{(V - V_{air}) \gamma_{air}}$$

$$= \frac{W_{total}}{(V_b + V_f + V_{FA} + V_{CA}) \gamma_w}$$

$$= \frac{W_{total}}{V_{solid} \gamma_w} \quad (\text{provided bitumen is considered as solid})$$



$$G_t = \frac{W_{total}}{\left(\frac{W_{CA}}{G_{CA}} + \frac{W_{FA}}{G_{FA}} + \frac{W_F}{G_F} + \frac{W_B}{G_B} \right) \frac{\gamma_w}{\gamma_w}}$$

(ii) Bulk specific gravity (G_m) / Actual specific gravity.

- It is the specific gravity of mix considering air voids also.

$$G_m = \frac{W_{total}}{V_{total} \cdot \gamma_w}$$

$$= \frac{W_{total}}{(V_{solid} + V_{air}) \gamma_w} \rightarrow (\text{Bitumen is considered as solid})$$

NOTE :- Since total volume $> V_{solids}$, hence $G_t > G_m$.

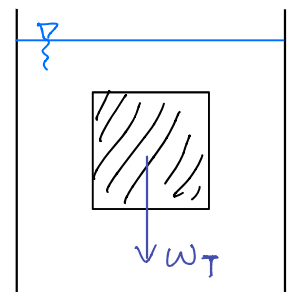
2) $W_T - F_B = W_w$ (Buoyant wt)

$$W_T - \gamma_w V_w = W_w$$

$$V_w = V_{mix} = V_T$$

$$W_T - V_T \gamma_w = W_w$$

$$V_T = \frac{W_T - W_w}{\gamma_w}$$



$$\therefore G_m = \frac{W_T}{V_T} = \frac{W_T}{\frac{W_T - W_w}{\gamma_w}} = \frac{W_T}{W_T - W_w} \gamma_w$$

$$G_m = \frac{W_T}{W_T - W_w}$$

W_T = total wt of mix / wt of mix in air

W_w = Submerged / Bouyant wt of mix / wt of mix in water.

(iii) AIR void percent % - (V_v)

- It is the % of air voids by volume in the mix

$$V_{av} = \frac{V_a}{V_T} \times 100$$

V_a = vol. of air
 V_T = Total volume

OR

$$V_{av} = \frac{G_T - G_m}{G_T} \times 100$$

NOTE:-

$$\begin{aligned} \frac{G_T - G_m}{G_T} &= \frac{\frac{W_T}{V_{solid} \gamma_w} - \frac{W_T}{V_{total} \gamma_w}}{\frac{W_T}{V_{solid} \gamma_w}} = \frac{\frac{V_{total} \gamma_w - V_{solid} \gamma_w}{V_{solid} \gamma_w}}{\frac{W_T}{V_{solid} \gamma_w}} \\ &= \frac{V_{air} \gamma_w}{V_{total} \gamma_w} = \frac{V_{air}}{V_{total}} \quad \checkmark \end{aligned}$$

(iv) % volume of bitumen (V_b) :-

- It is the volume of bitumen to the total volume of the mix.

$$V_b = \frac{V_{bitumen}}{V_{total}} \times 100 = \frac{\frac{W_B}{G_B \gamma_w}}{\frac{W_T}{G_m \gamma_w}} \times 100 = \frac{W_B}{G_b} \times 100 = \frac{W_B}{\frac{W_{CA} + W_{FA} + W_F + W_B}{G_m}} \times 100$$

(v) voids in Mineral aggregate % - (V_{MA})

- It is the total volume of voids in aggregates expressed in terms of total volume of mix.

- Here total vol. of voids consist of Volume of air voids and volume of bitumen.

$$V_{MA} = \frac{V_{voids}}{V_T} \times 100 = \frac{V_a + V_{bitumen}}{V_T} \times 100.$$

$$V_{MA} = \frac{V_a}{V_T} \times 100 + \frac{V_{bitumen}}{V_T} \times 100$$

$$V_{MA} = V_{av} + V_b$$

(vi) voids filled with bitumen (V_{FB}) $\Rightarrow$ (Equivalent to degree of sat.)

- It is the voids in mineral aggregates filled with bitumen.

$$V_{FB} = \frac{V_{\text{bitumen}}}{V_{\text{voids}}} \times 100 = \frac{V_{\text{bitumen}}}{\frac{V_T}{\frac{V_{\text{solids}}}{V_T}}} \times 100$$

$$V_{FB} = \frac{V_b}{V_{MA}} \times 100$$

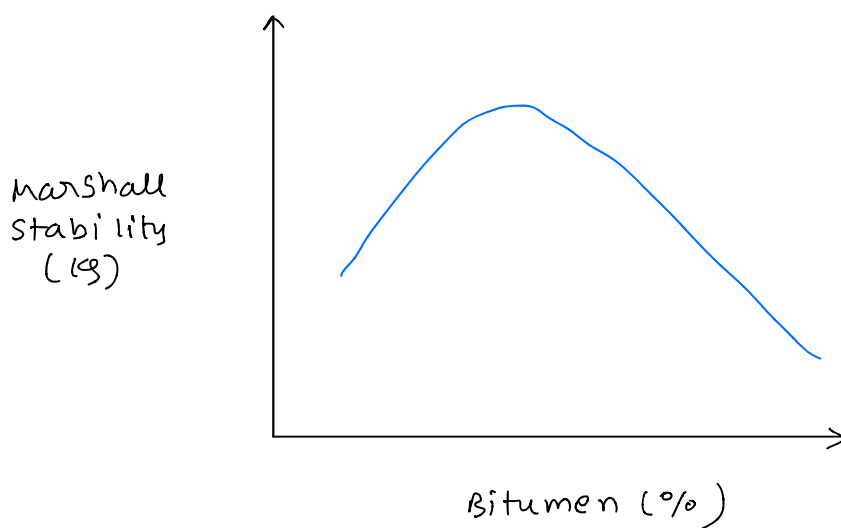
In marshall mix design

(a) marshall stability of a test specimen is the max load req. to produce failure when the specimen is pre heated to a prescribed temperature and load is applied at a const. strain of 50.8 mm/min.

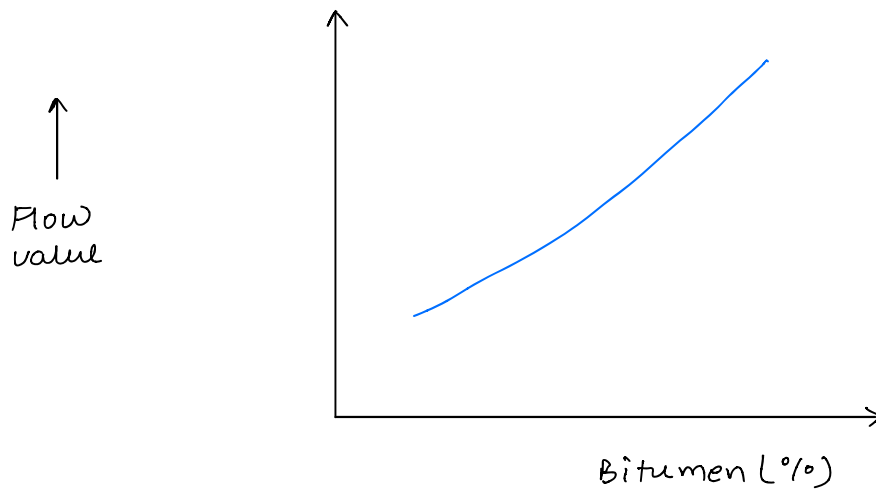
(b) while the stability test is in progress, dial gauge is used to measure the vertical deformation of the specimen. the deformation at the failure point in units of 0.25mm is called marshall flow value of the specimen.

— The avg of above properties are determined for each mix with different bitumen content and the following graphs are plot.

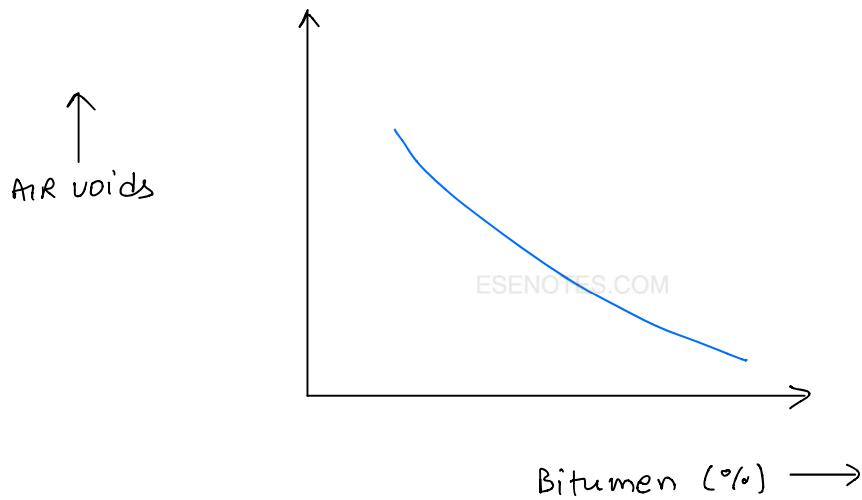
(a) BINDER CONTENT VS MARSHAL STABILITY :-



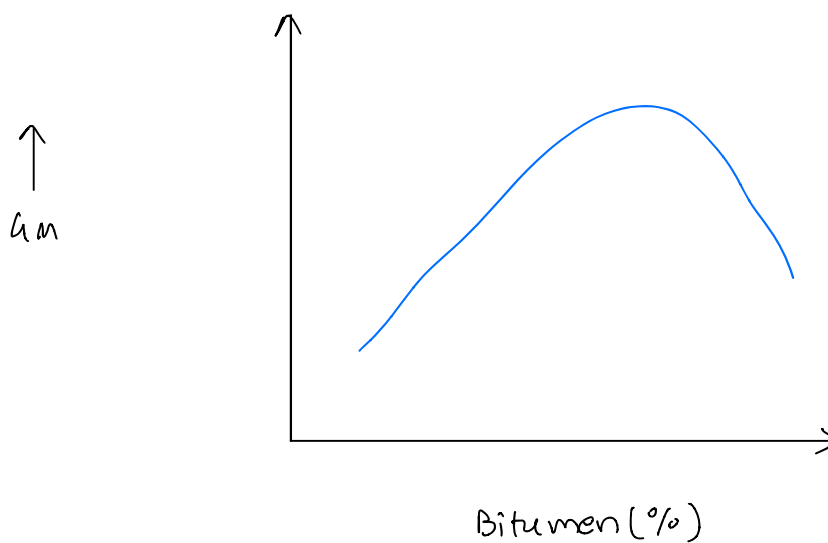
(b) BINDER CONTENT VS Marshall Flow %-



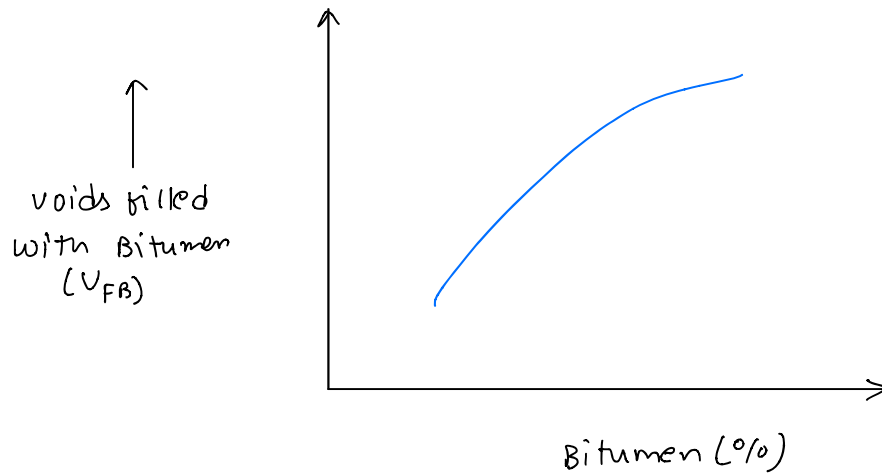
(c) BINDER CONTENT VS % Voids in total mix %-



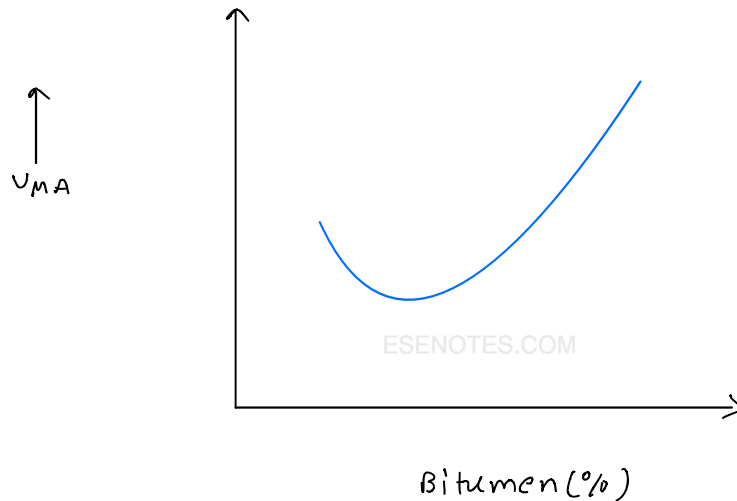
(d) BINDER CONTENT VS BULK SPECIFIC GRAVITY (G_m) %-



(c) BITUMEN CONTENT VS VOIDS FILLED WITH BITUMEN.



(d) BITUMEN CONTENT VS VOIDS in Mineral aggregate. (V_{MA})



— Finally the optimum binder content is determined for mix design by taking the avg of three bitumen content found from

- (a) STABILITY CURVE.
- (b) SPECIFIC GRAVITY CURVE.
- (c) % AIR VOIDS CURVE. (4%)

9 Mar

Q) A mixture contains CA with sp. gravity, $G_s = 2.7$, FA with $G_s = 2.9$ and mineral filler, $G_s = 1.5$ in proportion of 60:35:5 by weight. These materials when mixed with bitumen having sp. gravity 1.01 and compacted to a unit wt 2300 kg/m^3 contains 5% voids. How much bitumen does the specimen contain.

Souvik
2021

— How will you arrive at the optimum bitumen content based on curve plotted in marshall test.

Sol

$$\begin{aligned}c_A, \quad G_{CA} &= 2.7 \\G_{FA} &= 2.9 \\G_F &= 1.5 \\G_B &= 1.01\end{aligned}$$

let volume of the compacted mix be 1 m^3 .
wt v u u u u " $2300 \times 1 = 2300 \text{ kg}$.

$$W_{\text{mix}} = W_a^0 + W_b + W_F + W_{FA} + W_{CA}$$

let, W be the wt of CA , FA and filler.

$$\begin{aligned}2300 &= W_b + 0.05W + 0.35W + 0.6W \\2300 &= W_b + W \quad \text{--- (i)}\end{aligned}$$

$$V_{\text{mix}} = V_a + V_b + V_f + V_{FA} + V_{CA}$$

$$V_{\text{mix}} - V_a = V_b + V_f + V_{FA} + V_{CA}$$

$$0.95 = \frac{W_b}{1.01 \rho_w} + \frac{0.05W}{1.5 \rho_w} + \frac{0.35W}{2.9 \rho_w} + \frac{0.65W}{2.7 \rho_w}$$

$$950 = 0.376W + 0.99W_b \quad \text{--- (ii)}$$

From (i) & (ii) $W_b = 138.76 \text{ kg}$.

$$W = 2161.4 \text{ kg}$$

$$W_{CA} = 1297 \text{ kg}$$

$$W_{FA} = 756 \text{ kg}$$

$$W_F = 0.05 \times 2161.4 = 108.1 \text{ kg}.$$

Highway maintenance

- By early detection, inspection and repair of defects at initial stages, the rapid deterioration of pavement can be prevented.
- Maintenance of the highway is classified as.
 - (i) ROUTINE MAINTAINANCE :- filling up of pot holes, patch repairs, repair of shoulders, cross-slope repair of crack etc.
 - (ii) PERIODIC MAINTAINANCE :- renewal of wearing course etc.
 - (iii) SPECIAL REPAIRS :- overlays or upgrading of pavement.

DEFECTS IN PAVEMENTS :-

- Defects in the pavement are categorised as follows:-

(a) SURFACE DEFECTS :-

- These are associated with the surfacing layers and is due to excessive or deficit quantities of bitumen in these layers.
- All the surface defects are explained in next page.

DEFECTS ON PAVEMENT

JASPAL SIR

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SURFACE DEFECTS I. FATTY SURFACE

→ Non uniform spreading of cover aggregates.

- A liquid seal coat is applied to prevent it.



⊗ Binder comes to the surface due to excessive use of binder or deficiency of aggregate.

(Equivalent to bleeding in concrete)

→ Comes in top surface in hot weather.

→ Surface becomes slippery in cold weather.

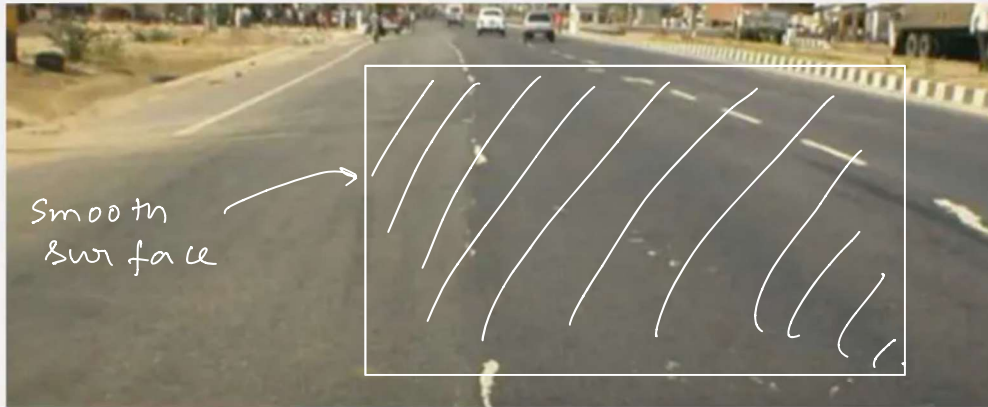
(ii) - A smooth surface, has a very low skid resistance and becomes very slippery when wet.

- It occurs due to polishing of aggregates under traffic and due to excessive binder use.

Treatment:-

Treated by resurfacing the road or by a porous carpet.

2. SMOOTH SURFACE



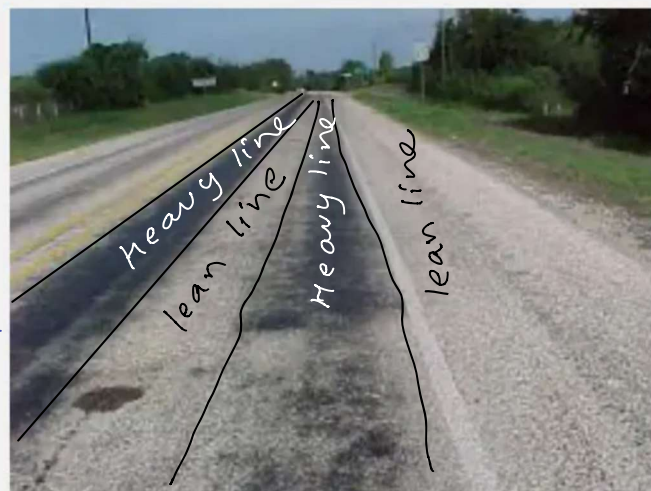
- It is characterized by the appearance of alternate lean and heavy lines of bitumen either in longitudinal or in transverse direction.

- It is formed due to mechanical faults or improper application of bitumen.

Treatment

In order to treat it new surface is provided after the removal of streaked surface.

3. STREAKING



- This defect is characterized by loss of aggregate from the surface and appearance of fine cracks.

- caused due to less use of bitumen in the surfacing
Treatment

- can be treated by providing seal coat.

4. HUNGRY SURFACE



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ALL SURFACE DEFECTS...

Surface Defects

Fatty surface

Smooth surface

Streaking Surface

Hungry surface

CRACKS are most common defects of bituminous surface and must be repaired as soon as possible, as it promotes water penetration in lower layer of pavement

CRACKS

1. HAIR LINE CRACKS

→ These cracks appears as short and fine cracks at close interval on the surface.

→ These are developed due to insufficient bitumen content & excessive filler
→ Improper compaction



→ These appear as inter connected crack forming a series of small blocks which resemble the skin of the alligator.

2. ALLIGATOR CRACKS / MAP CRACK or CRAZING.

→ It occurs due to excessive deflection of the surface over unstable subgrade.



→ It occurs along the length of the road. They appear more or less as a straight line along the road.

3. LONGITUDINAL CRACKS

- These cracks may appear either at joint b/w the pavement and the shoulder or joint b/w the two paving lanes.
- It occurs due to alternate wetting and drying beneath the poorly drained shoulder surface or due to weak joint b/w the two lanes.



- These cracks are formed parallel to the outer edge of the pavement usually (0.3-0.5)m inside from the edge.

→ These are caused by

(a) Lack of lateral support from shoulder.

(b) Settlement of underline material.

(c) Inadequate surface drainage.

(d) Frost heave.

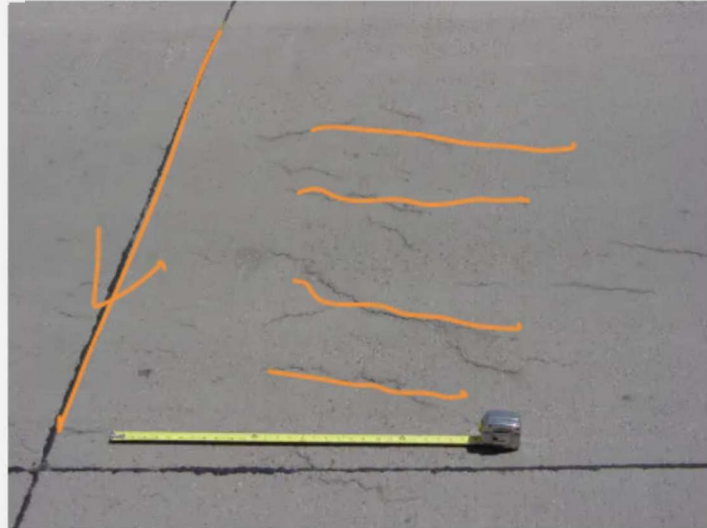
4. EDGE CRACKS



→ They appear in transverse direction or as inter connected crack forming a series of large blocks.

→ The primary cause is shrinkage of the bituminous layer itself with age.

5. SHRINKAGE CRACKS

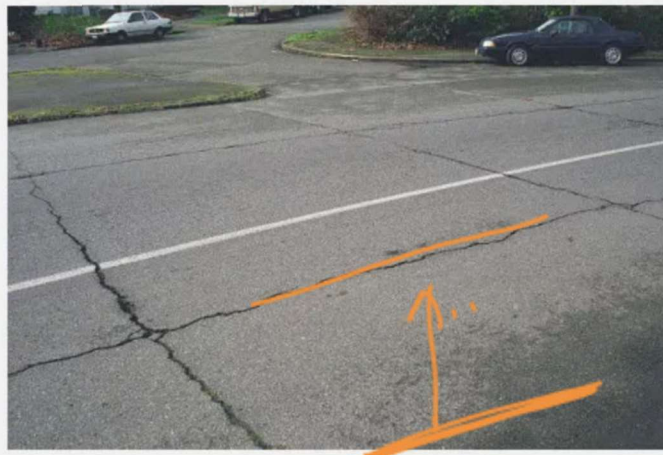


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→ The bituminous binder loses its ductility as it ages and becomes brittle.

6. REFLECTION CRACKS

→ These cracks appear in bituminous surface over joints and cracks in the pavement underneath.



→ These are due to joints and cracks in the pavement layer underneath.

NOTE * Treatment of crack

→ For all the cracks in pavement remain structurally sound then the cracks should be filled with bituminous binder having low viscosity.

- If there is any shape of pavement from its original shape, it is called as deformation.

DEFORMATIONS

1. SLIPPAGE

→ It is the relative layer & the layer beneath it.

→ It is characterized by the formation of accumulation of bitumen binder and formation of crack at a particular point in direction of thrust.



(a) Unusual thrust of wheels in particular direction

(b) Lack of bond b/w the surface & lower course

→ Its rectification consist of removing the surface layer around the affected area up to the point

where good bond b/w surface &

underneath layer exist

and patching of area with premix can be done.

2. RUTTING (in highway)

→ longitudinal depression or groove due to wheels of heavy vehicles.

→ The ruts are usually of the width of the wheel path.

→ It is caused due to following :-

- (i) Heavy channelized vehicle / Traffic
- (ii) Inadequate compaction of the mix at the surface.



→ It is the formation of fairly regular undulation across the bituminous surface.

3. CORRUGATIONS

- These are formed due to:-

- (a) lack of stability in mix
- (b) oscillation set up by the vehicle
- (c) faulty laying of surface.



It is rectified by removing the top surface and by providing the new premix.

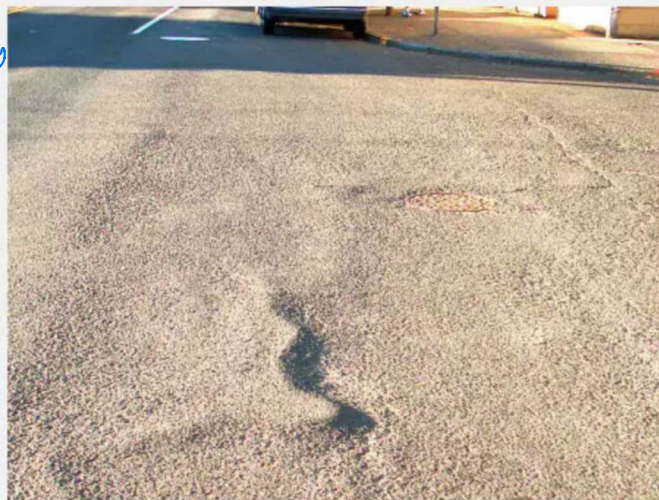
- Shoving is a plastic movement within the layer which results in bulging in the surface.

- localised bulging.

4. SHOVING

occurs due to

- (a) lack of stability in mix
- (b) lack of bond b/w bituminous surface & underlying layer.
- (c) Heavy traffic movement.



- To rectify it material is removed from affected area and stable premix patching is applied.

→ These are localised low areas of limited size below the desired profile in which water is normally collected.

5. SHALLOW DEPRESSIONS

- These are caused by settlement of lower pavement layers due to a pocket of inadequately compacted subgrade



→ These depressions are made up by filling with premix material

→ It is characterized by large deformations of the pavement
→ They are caused due to inadequate compaction of the fill or due to inadequate pavement thickness.

6. SETTLEMENT/ UPHEAVAL

If this indicates inherent weakness in the fill, it may be necessary to excavate the defective fill and redo the embankment



OR
→ Premix filling is done.

- If some defects are not rectified immediately it leads to disintegration (boreaking) of the pavement.

DISINTEGRATION I. STRIPPING

→ These defect is characterized by separation of bitumen adhering to the surface of aggregates in presence of moisture.

It is caused due to.

- (a) use of hydrophilic aggregates.
- (b) inadequate mix design
- (c) improper use of bitumen

LOSS OF AGGREGATE

→ It occurs in surfaces which have been provided with surface dressing.



It is treated by surface dressing.

→ The surface presents a round appearance in this case due to loss of aggregate.

2. RAVELLING

It occurs due to

- (a) Ageing of binder
- (b) loss of affinity of binder with aggregate.

Ravelling :-

→ Progressive failure of surface due to the failure of binder to the materials together. → relaying the pavement.



It can be rectified by applying "SEAL COAT"

- (a) Due to inadequate compaction
- (b) wet weather during construction
- (c) over heating of mix

March-11

It is a common defect in bituminous surface roads in which the edge is broken in irregular way and if not rectified in time, the surface may peel off in large chunk at the edges.

4. EDGE BREAKING/ FRAYED SURFACE

→ It is caused due to infiltration of water which weakens the base material.

→ Wornout shoulders having improper side support.



— The entire affected area is removed & relaying of the surface course is done.

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(5) POT HOLES :-

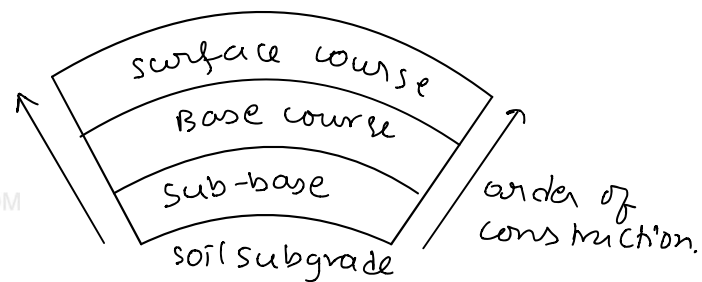
- These are bowl shaped holes of varying sizes in surface layer or extending upto the base course
- It is caused due to penetration of water in to the pavement, through the surface course.
- It may be caused due to lack of bond b/w the bituminous surface and the underlying aggregates or due to insufficient bitumen.
- It is rectified by filling the pot-holes with pre-mix or by penetration patching

PAVEMENT CONSTRUCTION

- The road pavement consists of following layers

(i) construction of soil subgrade :-

- Material used for construction of soil subgrade includes "Moorum" or "Gravel" or mixture of both, which are free from OM.



- Material used must have liquid limit (WL) < 50% and plasticity index (Ip) < 25%
- This material must be non-expansive. Eg- Black cotton soil is to be avoided.
- construction operation includes.
 - (a) Spreading of soil.
 - (b) Addition of water up to required proportion.
 - (c) Compaction by pneumatic or sheep foot roller.
 - (d) process is repeated by desired range of compaction.

NOTE :- As per IRC the specified compaction requirement of highway subgrade is 97% in terms of MDD.

(ii) CONSTRUCTION OF SUB-BASE :-

- Material used for construction of sub-base includes crushed stone, gravel, coarse sand, morum which is collectively termed as granular sub-base (GSB)
- Sub-base must have liquid limit $< 25\%$
plasticity index $I_p < 6\%$
CBR value $> 30\%$
- This layer also act as drainage layer.
- here construction operation includes :-
 - (a) Spreading of sub-base
 - (b) compaction by vibratory roller.
 - (c) process is repeated upto 98% of MDD.

(iii) Construction of Base course :-

- It is constructed using WMM (Wet Mixed Macadam). which consist of well graded hard crushed aggregates and adequate proportion of water mixed thoroughly in a mixing plant.
- The prime function of base course is to provide load distribution and surface drainage.
- construction operation includes.
 - (a) Preparation of WMM.
 - (b) Spreading.
 - (c) compaction by vibratory roller.

NOTE :- Instead of using WMM, WBM (Water Bound macadam) can also be used.

- the basic difference b/w WMM, WBM are as follows :-

WMM

- (i) Stone aggregates and binding materials are pre-mixed in a batching plant
- (ii) It is prepared by stone aggregates and binders
- (iii) Stone aggs are comparatively smaller in size (4.75-20)mm

WBM

- (i) Stone aggregates, screening material and binders are overlaid one after the another.
- (ii) It is prepared by stone agg, screening and binding materials.
- (iii) Stone aggs are bigger in size (45-90)mm

(iv) In this case water is premixed.

(v) In this case water is sprinkled on dry mix.

(iv) CONSTRUCTION OF SURFACE COURSE :-

(a) PRIME COAT :- In this case application of low viscous liquid over an existing porous pavement surface like WBM is done.

- The main objective is to fill capillary voids and to bind the loose material with each other.
- Generally tar, low viscous bitumen or cutback bitumen are used in this case.
- Primed surface is allowed to cure for at least 24 hrs during which no traffic is allowed.

(b) TACK COAT :- It is the application of bituminous material over an existing pavement surface which is relatively impervious surface.

- It can also be applied to pervious surface like WBM which are already being treated by prime coat.
- Here viscosity is comparatively more than prime coat.
- It provides bonding b/w the two layers.

(c) SEAL COAT :-

- It is usually recommended as a top coat over certain bituminous pavement which are not impervious, such as pre-mixed carpet.
- It can also be provided over worn out existing bituminous pavement.
- Its main function is to drain out water and to provide smooth surface.

(V) SURFACE DRESSING :-

- It is provided over an existing pavement to serve as bearing course.
- The single coat surface dressing consist of single application of bituminous binder or it may be followed by spreading of aggregates.
- Its main function are.
 - (a) To serve as a wearing coat.
 - (b) To drain off the water from the surface.
 - (c) To provide mud free / dust free pavement surface.

NOTE :- PENETRATION MACADAM :-

- It is used as base of binder course.
- The coarse aggregate are first spreaded and compacted well in dry state and then the hot bituminous binder is spread over it.
- The bitumen penetrate into the voids from the surface of compacted aggregate & fill up a part of void and bind the aggregates.

NOTE :- PREMIXED MTD :-

- In this method the aggregates and bituminous binder are mixed thoroughly before spreading and compacting.
- Depending upon gradation of aggregates premixed construction are of following types.

- | | | |
|---|------------|----------------|
| ① | Bituminous | macadam. |
| ② | " | premix carpet. |
| ③ | " | concrete. |
| ④ | Sheet | Asphalt. |
| ⑤ | Mastic | Asphalt. |

PAVEMENT EVALUATION & STRENGTHENING OF EXISTING PAVEMENT

- Pavement evaluation involves study of various factors that governs the stability of the existing pavement like subgrade support, pavement composition and its thickness, traffic loading, environment condition.

- Pavement evaluation is carried out for following aspects:-

(a) STRUCTURAL EVALUATION.

(b) Evaluation of pavement surface condition.

- Structural evaluation is done by CBR method or by Benkelman Beam mtd, whereas evaluation of pavement surface is done by bump integrator, profilograph or profilometer.

- If after evaluation following condition prevails then

(a) If pavement is structurally stable then only repair can be carried out

(b) If pavement is structurally unstable, then periodic maintenance work and strengthening of pavement is required.

(c) If the existing pavement is completely deteriorated an overlay won't work and it should be completely replaced.

- Strengthening may be done by providing additional thickness of the pavement in one or more layers termed as overlay which are of following types:-

(a) Flexible overlay over flexible pavement.

(b) " " " rigid pavement

(c) Rigid " " rigid "

(d) " " flexible "

- The design of these overlay can be done by either of two.

(a) CBR method

(b) BENKLEMAN BEAM MTD.

CBR Method :-

- In CBR method test is performed over the existing pavement to find the CBR value and corresponding thickness of pavement required.

- The thickness of the overlay is then computed by subtracting the thickness of the existing pavement with the thickness of pavement required.

Benkleman Beam Method :-

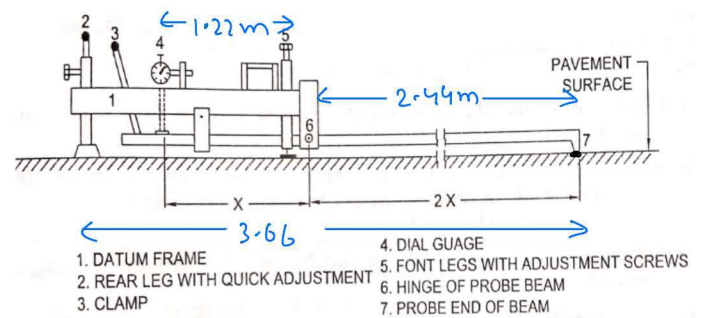
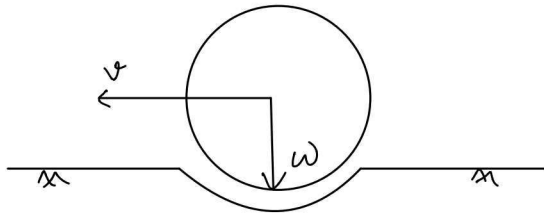


Fig. 10.11 Benkelman beam

- A well compacted pavement section deforms elastically under wheel load (bowl shaped deflection).
- when the wheel load application moves away there is elastic recovery or rebound deflection of the deformed pavement.
- The amount of deflection depends upon following
 - (a) wheel load (b) pavement thickness
 - (c) soil strength (d) Surface temperature.
- larger the rebound deflection weaker is the pavement structure which require higher overlay thickness.
- To perform the test "Benkelman Beam" of length 3.66m is used which is pivoted at distance of 2.44m from contact point. Hence if deflection of Δ is observed at point of contact, it is recorded as $\Delta/2$ in dial gauge.
- A min of 10 points are selected along the outer wheel path of each lane.
- A single rear axle truck is used having load of 81.7 kN. & tyre pressure of 0.56 MPa/m² (5.6 kg/cm²).
- The probe is inserted in b/w the two wheels, and the wheel position represents the point where deflection is desired.

Souvik 2021 - The truck is moved forward slowly & dial gauge reading is taken when truck is 2.7m and 9m away from the initial position.

- Temp. of pavement is also recorded.
- The intermediate and final reading are subtracted from the initial reading.
- If the deflection obtained compared with 0.025mm, the actual deflection is twice the diff b/w initial and final reading.
- once atleast 10 readings of deflection are obtained its analysis is carried out as follows.

(a) compute mean deflection, $\bar{D} = \frac{\sum D_i}{10}$ ($n \geq 10$)

(b) compute standard deviation, $\sigma = \sqrt{\frac{\sum (D_i - \bar{D})^2}{n-1}}$

(c) compute characteristic deviation,

$$D_c = \bar{D} + \sigma.$$

(d) compute overlay thickness for this deflection.

$$(cm) \leftarrow h_o = 550 \log_{10} \left(\frac{D_c}{D_a} \right)$$

D_a = allowable deflection

= 1 mm for design traffic, $A = 1500 - 4500$

= 1.25 mm " " " $A = 450 - 1500$

= 1.5 mm " " " $A = 150 - 450$

Here design traffic $A < P(1+r)^{n+10}$