1 GEOMETRIC DESIGN

It deals with the visible elements of the Road.
Various Geometric design components depends on

1) Types of Roads

A) Rural Roads

i) Expressway → speed upto 120 KMPH
ii) National Highway → joins various states
iii) State Highways → joins various Districts
iv) Major District Roads → joins Areas of population or production with Main highway
(v) Other District Roads → joins Rural Areas with Markets
vi) Village Roads → joins various Villages

NOTE IRC 73 deals with the Geometric Design of Rural highways

B) URBAN Roads

i) expressway → 120 KMPH (Devided Arterial)
ii) Arterial Roads → 80 KMPH
iii) Sub-Arterial Roads → 60 KMPH
iv) Collector streets → 50 KMPH
v) local streets → 30 KMPH

2) Type of vehicle

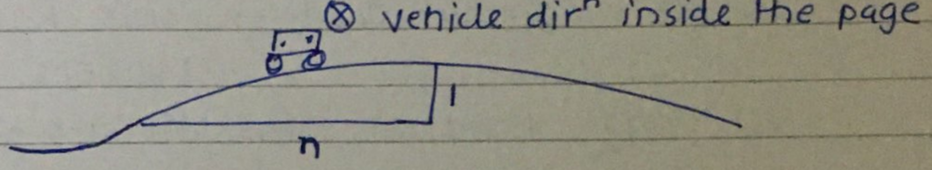
The vehicle for which the Road elements are designed are called design vehicle
Length, Height, Width of Designed vehicle are used as design parameter for the Roads
IRC:003

Width of vehicle → 2.5 m
Height of vehicle → 3.8 to 4.75 m → Height of Double Decker BUS.

3) Topography

It is classified on the basis of general country slop accross the Road alignment

It is expressed as 1 in n or x %
x % = x in 100



cross country slop	class
0 – 10 %	plain
10 – 25 %	Rolling
25 – 60 %	Mountaneous
> 60 %	steep

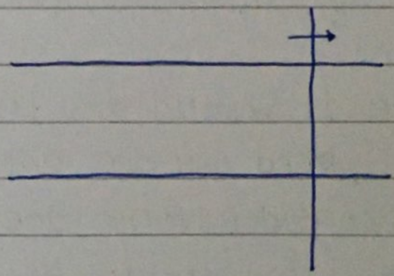
If cross slop of the country is large then large expenditure has to be made in Altering the Alignment for design speed to provide a larger Radius of curve to counter act against centrifugal force which causes skidding or overturning problems.

Hence when cross country slop is large the velocity should be restricted

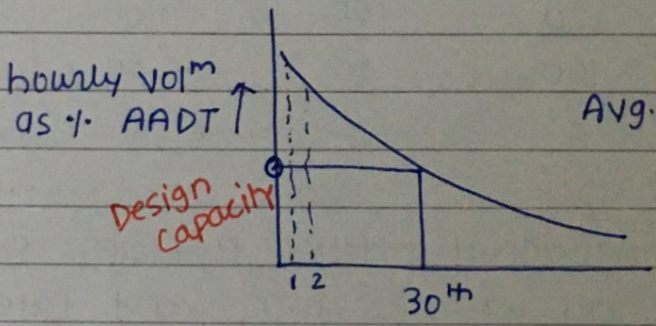
4) Traffic capacity:- It is the ability of Road to accomodate Max. Traffic volume. (at a particular LOS.)

Traffic volume is no. of vehicles crossing a point or section on the road in unit time.

Capacity and volume are both expressed in veh/Hr.



Normally design capacity is taken as 30th highest hourly volume



AADT
⇓
Avg. Annual Daily traffic
⇓
$\frac{\text{Yearly Traffic}}{365}$

Generally 30th highest hourly volm for Indian conditions comes around 8-10% AADT
for ex. if AADT = 2000 veh/day the design capacity or 30th highes hourly volume will be around 160 - 200 veh/Hr

Depending upon traffic capacity width of Road is decided.

5) Design speed :- It is theortically decided as 98th percentile speed , That is the speed at or below which 98% of vehicles are moving.

However from economical point of view IRC has Limited the design speed based on Topography

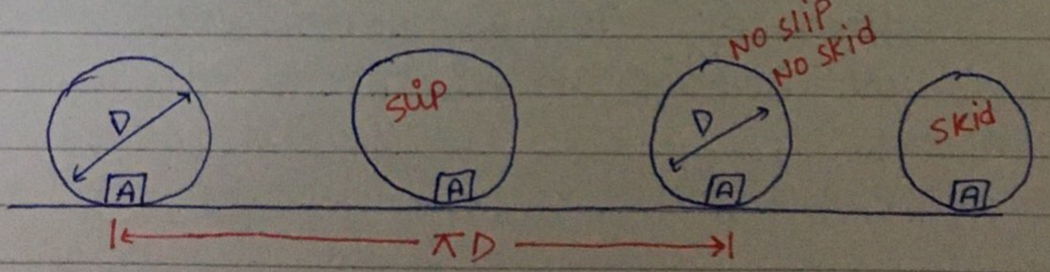
• Normally Ruling speed should be the Guiding criteria However Min. design speed can be adopted in localised sections where cost considerations does not permit Ruling speed.

	Plain		Rolling	
	Ruling	Minimum	Ruling	Minimum
Express way	120	100	100	80 KM/H
NH/SH	100	80	80	65

6) Surface charecteristics :-

A) Friction :- Longitudnal friction cofficient as recommended by IRC → 0.35 - 0.4 and lateral or Transverse friction cofficient as recommended by IRC is 0.15

lack of friction causes slipping or skidding.

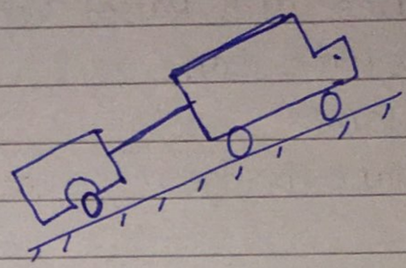


if one revolution of wheel leads to longitudnal movement

less than πD → slipping

more than πD → skidding

B) Unevenness index :- This index is a cummulative Measure of Vertical undulations per unit length of the road. It is Measured using Bump Integrator



Classification of Roads Based on unevenness index

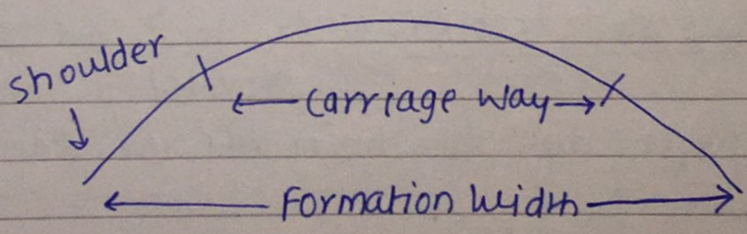
1) UI < 1500 mm/km → Good surface
2) UI upto 2500 mm/km (100 kmph speed) → Satisfactory surface
3) UI > 3200 mm/km (55 kmph speed) → unsatisfactory surface

Various Geometric Design components of Highway

1) cross-sectional Element
2) sight Distance
3) Horizontal alignment details
4) Vertical alignment details
5) Intersection detail

① cross-sectional Elements:-

a) carraige Way :- It is the part of pavement designed to carry vehicles.



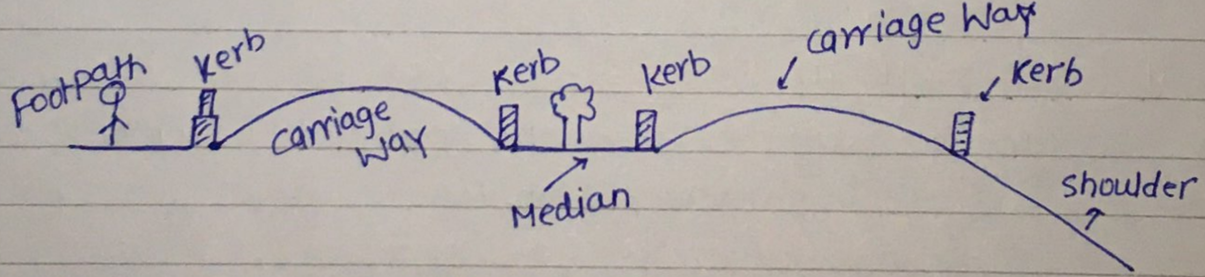
Type of Road	Carriage Way Width
single lane	3.75 m
Two lane with No kerb	7 m
Two lanes with Raised kerb	7.5 m
Intermediate Lane	5.5 m
Multi lane	3.5 m / Lane
Multi lane bridge	3.5 m / Lane + 0.5 m per carriage Way

b) **shoulder :-** shoulders are provide to accomodate stopped vehicles and to provide lateral confinement to the pavement layers

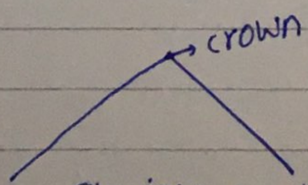
Desirable width of shoulder is 4.6 m with a Min. of 2.5 m for 2 lane Rural highway

NOTE → formation width for single lane / two lane NH section is 12 m as per IRC

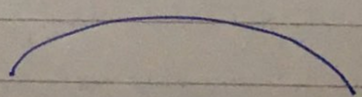
c) **kerb :-** It indicates the boundary b/w pavement and shoulder or Median or Footpath



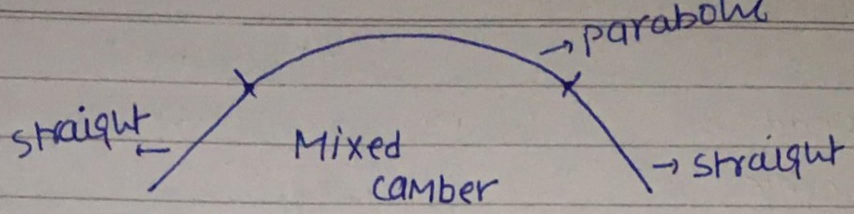
d) **camber or cross-slope / cross Fall :-** It is the slop provided to the road surface in transverse dir^n to drain off the Rain-water



Straight camber

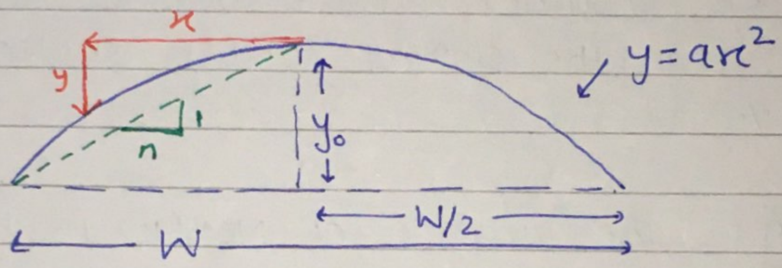


parabolic camber



For slow moving traffic straight camber can be adopted but for High speed traffic where crown has to be crossed frequently during overtaking, parabolic camber is preferred.

EQ^n of parabolic camber



From similar Δ $\quad \frac{1}{n} = \frac{y_0}{W/2} \qquad y_0 = \frac{W}{2n}$ —(i)

at $x = W/2$, $y = y_0$

$$y_0 = a\left(\frac{W}{2}\right)^2 \quad \text{—(ii)}$$

From ① & ②

$$a\left(\frac{W}{2}\right)^2 = \frac{W}{2n}$$

$$a = \frac{2}{nW}$$

Hence Eq^n of parabola is $\boxed{y = \frac{2}{nW} x^2}$

In field camber is checked by camber boards / Templates

NOTE Bitumineous pavement → Parabolic camber

Rigid pavement → straight camber

IRC Reccomandations for camber :-

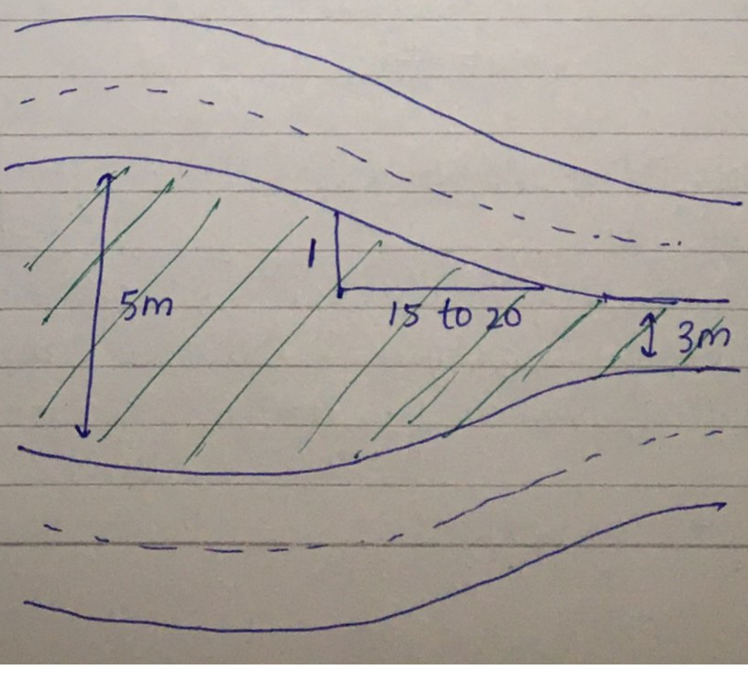
Type of Road	Rainfall Low (<100 cm)	Rainfall Heavy (>100 cm)
cement concrete or	1.7%	2%
High type Bituminous pavement	~~2%~~	
WBM / Gravel	2.5%	3%
Thin Bituminous pavement	2%	2.5%
Earthen	3%	4%

NOTE slope of shoulder should at least be 0.5% steeper than the slop of camber subjected to a Min. of 3%

e) Median :- The purpose of Median is to prevent head on collission of vehicles. It is also known as traffic seprator

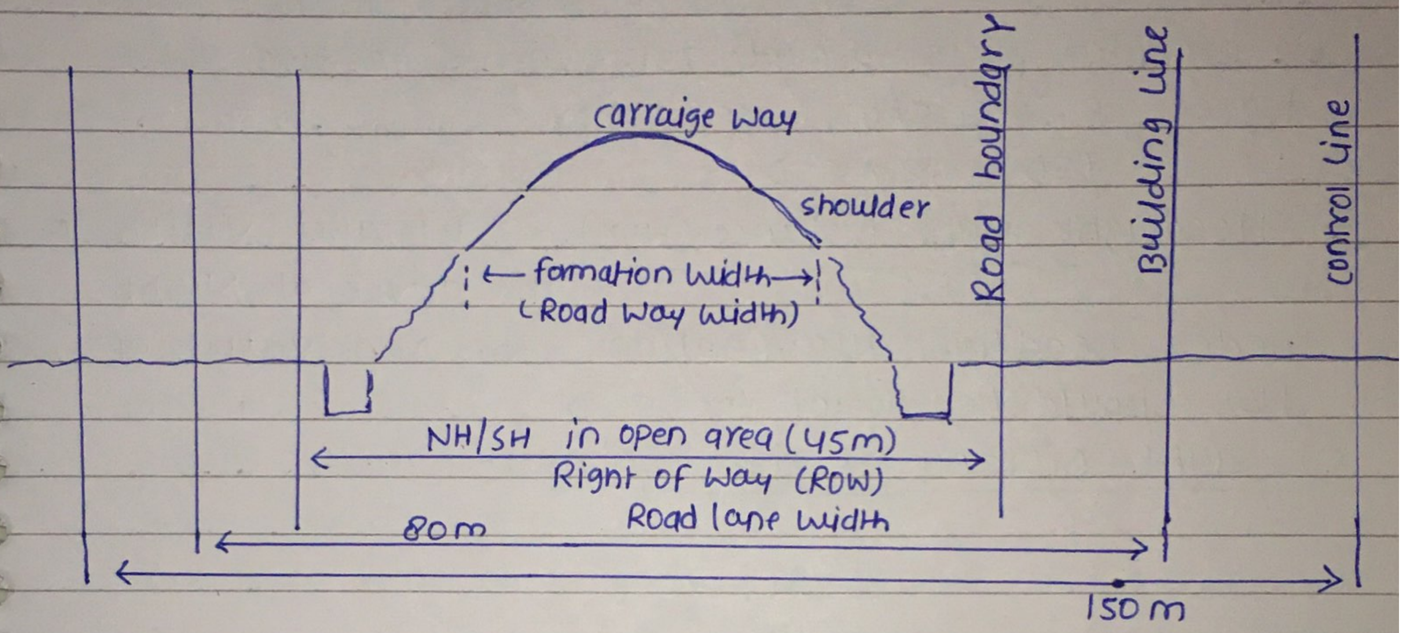
Min. desirable width for Rural highways is 5 m and if lane width is restricted then the value maybe reduced to 3 m

Width of Median for bridges should be b/w 1.2 – 1.5 m



Transition in Median should be 1 in 20 to 1 in 15

f) Road Margins :-



Building line → Represents the Road width upto which No Building Activity is permitted

control line → Represents distance upto which Nature of Building is controlled

2) sight Distance :- Geometric design of highway is done in such a way that from every point on highway the length of view available is sufficient so that the vehicle could be stopped in that visible distance or operations like overtaking could be safely performed.

Various sight distances are :-

1) stopping sight Distance, SSD :- It is also known as absolute Min. sight distance or non-passing sight distance.

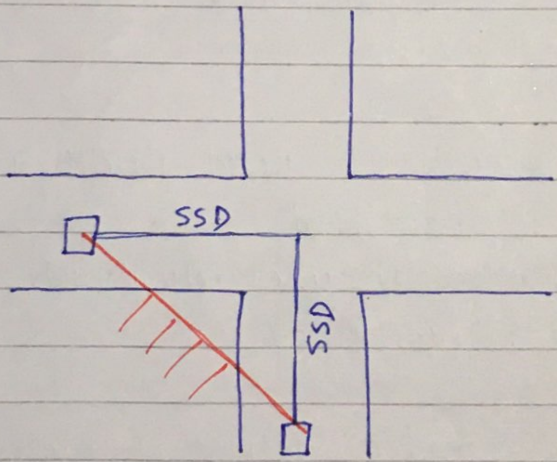
2) Overtaking Sight Distance (OSD) :- It is also known as passing sight Distance

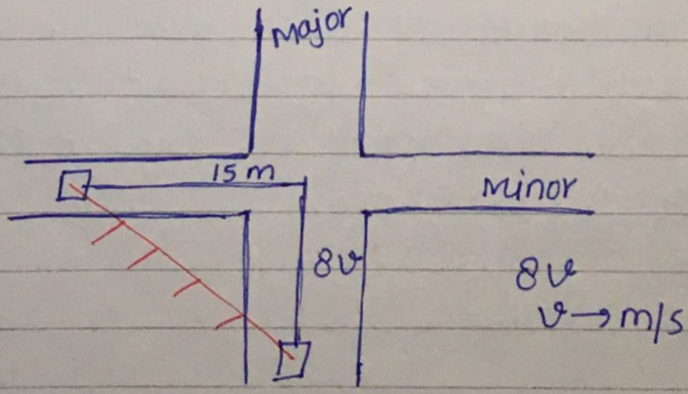
3) Intermediate Sight Distance (ISD) :- When overtaking sight Distance can not be provided, we provide ISD so as to give some degree of overtaking opportunity

$$ISD = 2 \times SSD$$

4) Headlight Sight Distance (HSD) :- Distance visible to the driver at Night under headlight illumination, the Min. value of HSD should be equal to SSD

5) Sight Distance at Intersections :-





At priority intersections (where a Major Road crosses a Minor Road) the sight Tringle is formed by providing a Min. visibility of of 15m along the Minor Road and 8 sec travel distance along the Major Road (v in m/s)

Stopping sight Distance :- SSD.

It is the Min. sight distance (visibility) that should be available from all spots on highway so that vehicles travelling at design speed could be safely stopped within that distance.

SSD depends on :-

1) Reaction time of Driver :- IRC recommends 2.5 sec. as the reaction time for SSD calculation

2) Speed of vehicle

3) Brake efficiency :- IRC assumes brake efficiency as 50%. It has already been included in longitudnal friction cofficient recommended by IRC

4) Friction cofficient of Road (longitudnal) :- As per IRC 0.35→0.4

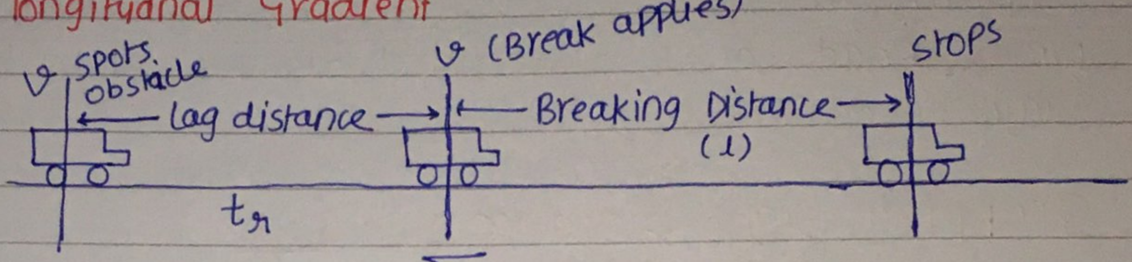
Speed (kmpH)	μ
≤ 30	0.4
60	0.36
≥ 80	0.35

5) longitudnal Gradient of Road :- Up Gradient will lead to a lower value of SSD and Down Gradient will lead to a higher value of SSD

$v \rightarrow m/s$
$V \rightarrow KmpH$

Calculation of SSD :-

i) When the vehicle is moving on level Road that is No longitudnal Gradient



$\overleftarrow{f} = \mu mg$

$a = -\mu g$

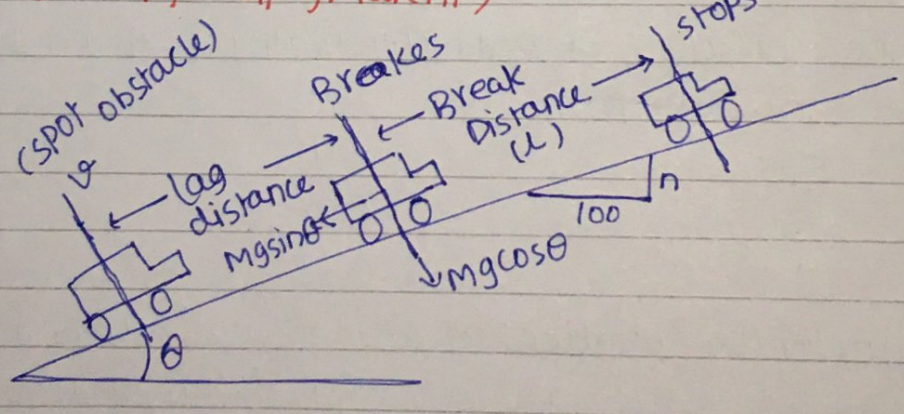
Lag distance $= v t_r$

Breaking distance (l)

$$v^2 = u^2 + 2as$$
$$0 = v^2 - 2\mu g l$$
$$l = \frac{v^2}{2\mu g}$$

$$\boxed{SSD = v t_r + \frac{v^2}{2\mu g}}$$

ii) When the vehicle is moving on a longitudnal Gradient (say on n% up gradient)



$$f = \mu mg\cos\theta + mg\sin\theta$$
$$a = -(\mu g\cos\theta + g\sin\theta)$$

lag distance $= v t_r$

Braking distance (l)

$$v^2 = u^2 + 2as$$

$$0 = v^2 - 2(\mu g \cos\theta + g\sin\theta) l$$

$$l = \frac{v^2}{2g(\mu\cos\theta + \sin\theta)}$$

$$l = \frac{v^2}{2g\cos\theta(\mu + \tan\theta)}$$

for small θ

$$l = \frac{v^2}{2g(\mu + 0.01n)}$$

SSD = lag distance + Braking distance

$$SSD = v t_r + \frac{v^2}{2g(\mu + 0.01n)}$$

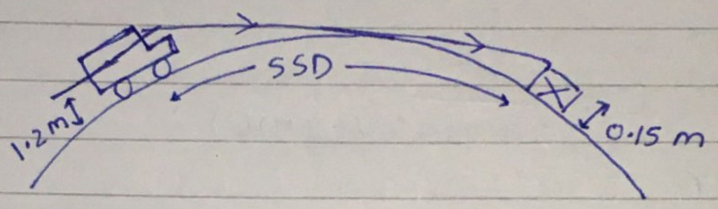
+ → up gradient
− → down gradient

v → Design speed in m/s
t_r → reaction time (2.5 sec)
μ → longitudnal friction cofficient (0.35 − 0.4)
n% → longitudnal Gradient

IRC Recommendations :- i) on a single lane Road with Two way traffic the Min SSD should be equal to twice of SSD (for same speed)

ii) For undevided highway with two way traffic condition the effect of Gradient is not considered in SSD calculations However on devided Highways effect of Gradient should be considered.

SSD on vertical surves should be the length along centre line of curv from which a driver with an eye level of 1.2 m can spot top of an obstacle 0.15 m above ground



If SSD can not be provided in a perticular strech of Road, proper sign boards with speed restrictions Must be provided.

Que. calculate safe SSD for a design speed of 50 KMPH on i) two-way traffic on a two-lane Road.
ii) two-way traffic on a single lane Road.

$V = 50$ KMPH , $v_i = 13.88$ m/s

i) $SSD = vt_r + \frac{v^2}{2g\mu}$

$= 13.88 \times 2.5 + \frac{13.88^2}{2 \times 0.36 \times 9.8}$

$= 62.025$ m

ii) $SSD = 2 \times 62.025$

$= 124.05$ m.

Que. Driver of a vehicle travelling at 60 KMPH on an upgradient requires 9m less to stop after applying the brakes. as compare to a driver travelling down the slop at same speed.
What is the Gradient of Road.

$V = 60\text{ KMPH}, v = 16.67 \text{ m/s}$

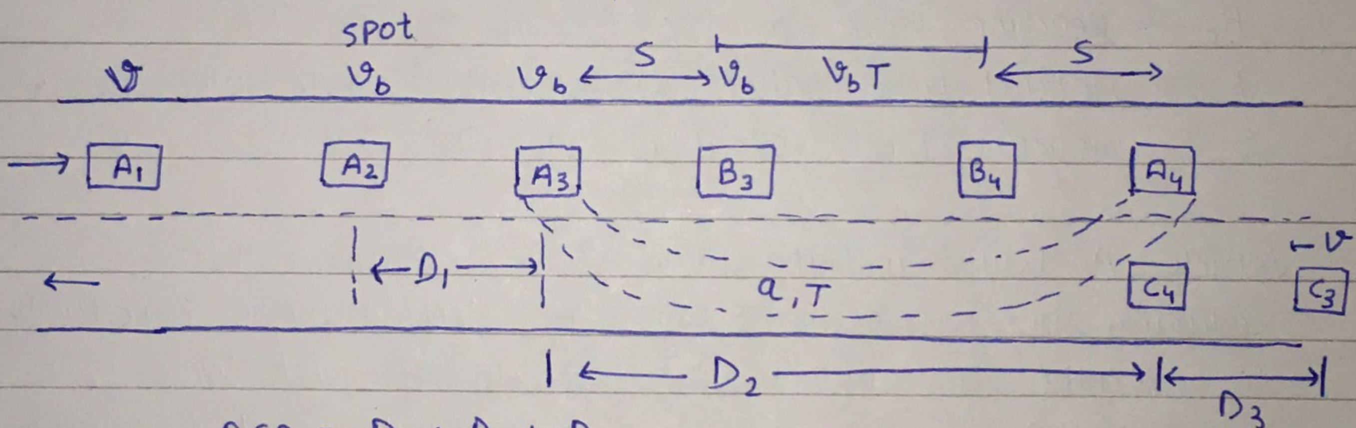
$$\frac{v^2}{2g(\mu - 0.01n)} - \frac{v^2}{2g(\mu + 0.01n)} = 9$$

$$\frac{16.67^2}{2 \times 9.8(0.36 - 0.01n)} - \frac{16.67^2}{2 \times 9.8(0.36 - 0.01n)} = 9$$

$n = 4.066\ \%$

OVERTAKING SIGHT DISTANCE :-

It is provided for safe overtaking operations.



$OSD = D_1 + D_2 + D_3$

$D_1 = v_b t_r$

$D_2 = 2S + v_b T = v_b T + \frac{1}{2} a T^2$

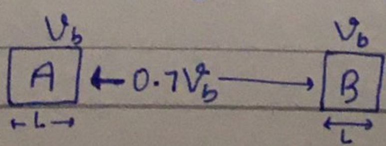
$T = \sqrt{\frac{4S}{a}}$

$D_3 = vT$

if the speed of slow-moving vehicle is not given it can be taken as $v_b = v - 4.5$ (m/s)

$V_b = V - 16$ (km/H)

spacing b/w vehicles $S = 0.7 v_b + L$



$(v_b \rightarrow \text{m/s})$

L → 6m assume (if Not given)

$S = 0.2V + L$ (V – KMPH)

IRC Recommendations for acceleration :-

V_b (KM/H)	a (m/s^2)
80	0.72
100	0.53

A → represents overtaking vehicle
B → overtaken vehicle
C → vehicle coming from opposite Dir^n
v → Design speed (m/s)
v_b → speed of overtaken vehicle (m/s)
t_r → Reaction time in sec. (2 sec as per IRC)
T → Actual time taken in overtaking Maneuver (in sec)
S → Distance from c/c b/w vehicles A & B

Vehicle A with initial speed v is moving after reducing it's speed to v_b and looking for an opportunity to overtake a slow moving vehicle B. At some instance (A_2), A spots an opportunity But persiving the opportunity some time is lost in reaction (2 sec) and in this time vehicle A moves by a distance D_1, But the moment A starts accelerating it may find that the opportunity to overtake is no longer available Hence it may remain moving behind vehicle B maintaining a distance S at a lower speed v_b. if However the vehicle A finds that the opportunity is still available it will accelerate moving into the adjacent lane and again come back to it's initial lane maintaining the same distance S as before

The distance moved by A in actual overtaking maneuver is D_2 in time T, in this time the ~~vehicle~~ distance moved by vehicle coming from opposite lane is D_3

NOTE if the spacing b/w vehicle A & B is not same say (S_1 and S_2)

$$D_2 = (S_1+S_2) + V_b T = V_b T + \frac{1}{2} a T^2$$

$$T = \sqrt{\frac{2(S_1+S_2)}{a}}$$

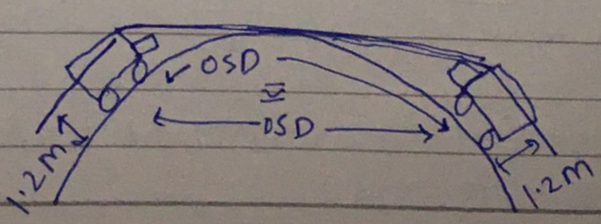
IRC Recommendations :-

(i) on devided highways and for roads with one way traffic regulations OSD is taken as $D_1 + D_2$

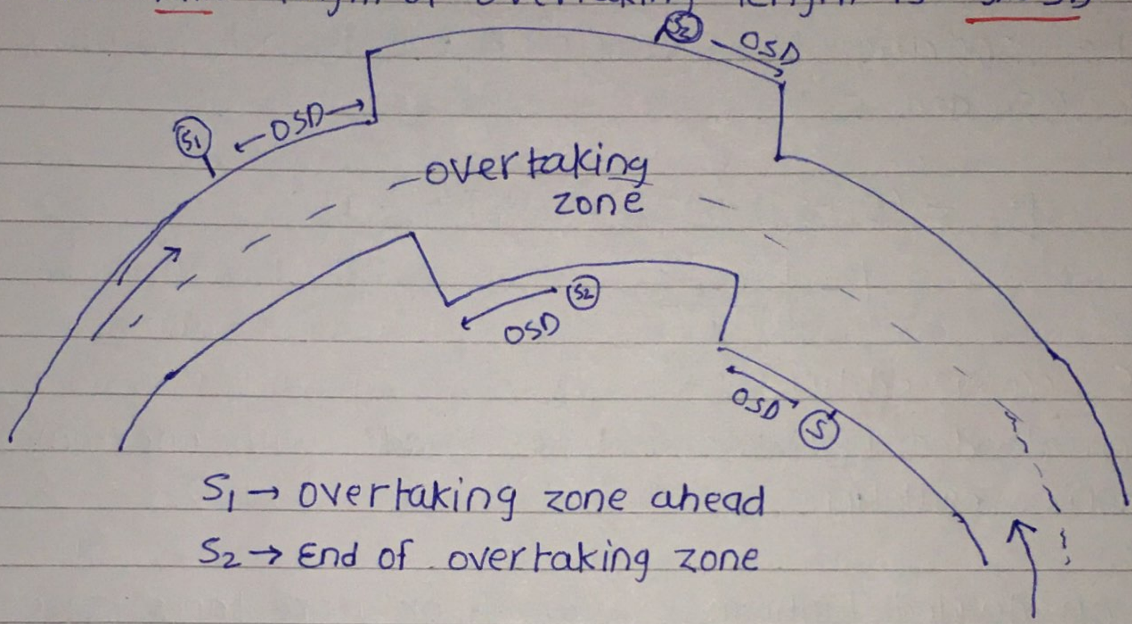
(ii) on devided highways with 4 or more lanes IRC suggests that there is no need to provide OSD But SSD should always be provided

NOTE Effect of Gradient is not considered in OSD calculations (However Gradient tends to increase the OSD slightly)

(iii) on vertical curve OSD should be along the center line of the curve ~~from~~ of Road from which a driver with it's eye level at 1.2 m ~~can b~~ about road surface and can see the top of object 1.2 m



iv) if OSD can't be provided throughout the length of the Road we provide overtaking zones at certain intervals.
Desirable length of overtaking zone is 5 x OSD and Min. length of overtaking length is 3 x OSD



$S_1 \rightarrow$ overtaking zone ahead
$S_2 \rightarrow$ End of overtaking zone

Que. On a two way traffic Rode the speed of an overtaking veh. and overtaken veh. are 65, 40 KMPH respectively if the avg. acceleration is 0.92 m/s^2 Determine the OSD ?

$V = 65$ KMPH , $v = 18.05$ m/s
$V_b = 40$ kmph , $v_b = 11.11$ m/s

$D_1 = v_b t_r = 11.11 \times 2 = 22.22$ m

$S = 0.7\left(\frac{5}{18} \times 40\right) + L = 13.77$

$T = \sqrt{\frac{4S}{a}} = \sqrt{\frac{4 \times 13.77}{0.92}} = 7.74$

$D_2 = 2S + v_b T = 2 \times 13.77 + 11.11 \times 7.74$

$D_2 = 113.56$ m

$D_3 = VT = 18.05 \times 7.74 = 139.75$

$OSD = D_1 + D_2 + D_3 = 275.53$ m.

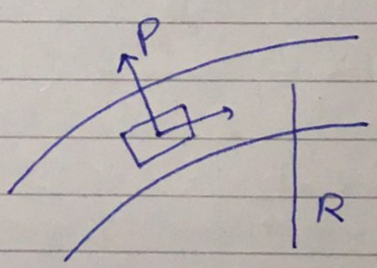
HORIZONTAL ALIGNMENT DESIGN :-

Design elements of Horizontal Alignment are :-

1) Radius of circular curve
2) Superelevation
3) Extra widening at Horizontal curve
4) Design of Transition curve
5) set back distance

- Design speed is the single Most important factor in the design of Horizontal Alignment

presence of Horizontal curve imparts centrifugal Force (P)



$P = \frac{mv^2}{R}$

$\frac{P}{mg} = \frac{v^2}{gR}$ centrifugal Ratio or impact Ratio

$v \rightarrow$ Design speed , $R \rightarrow$ Radius of curve

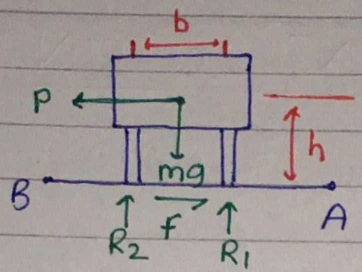
on Horizontal Road surface the centrifugal force generated is counter acted by transverse Friction

centrifugal force has two effects

1) Transverse skidding
2) overturning about the outer wheel

Condition for No overturning / Skidding.

Case (i) When the Road surface is Flat i.e No cross slop



$N = R_1 + R_2 = mg$

A) For No overturning

In the limiting case when vehicle is just about to overturn about the outer wheels ($R_1 = 0$), Balancing moment about the outer wheel we have

$$Ph = mg\,(b/2)$$

Hence for no overturning we should have

$$Ph \leq mg\left(\frac{b}{2}\right)$$

$$\frac{P}{mg} \leq \frac{b}{2h}$$

$$\boxed{\frac{v^2}{gR} \leq \frac{b}{2h}}$$

B) For No skidding

In the limiting case when the Veh. is just about to skid ($f = \mu mg$), Balancing forces we've

$$P \leq \mu mg$$

Hence for No skidding we should have

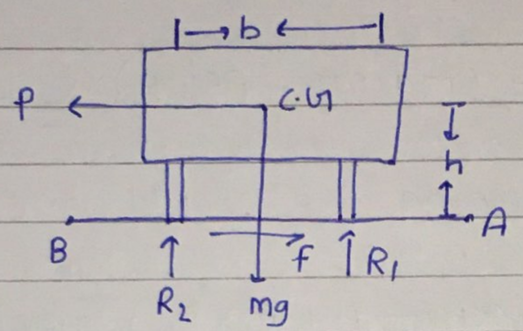
$$P \leq \mu mg$$

$$\frac{P}{mg} \leq \mu$$

$$\boxed{\frac{v^2}{gR} \leq \mu}$$

if $\mu \le \frac{b}{2h}$, skidding occurs before overturning

if $\mu > \frac{b}{2h}$, overturning before skidding.



Balancing moment about center of Gravity

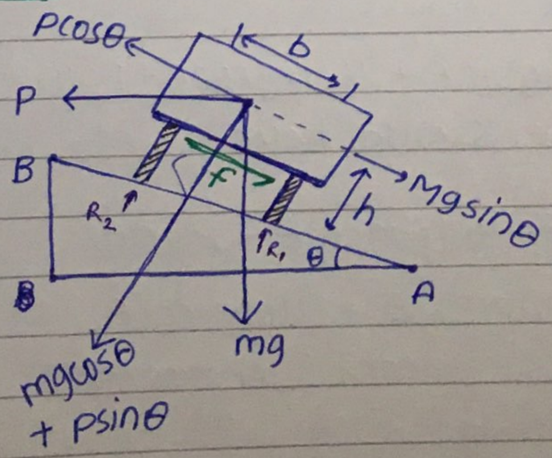
$$R_2\left(\frac{b}{2}\right) = R_1\left(\frac{b}{2}\right) + fh$$

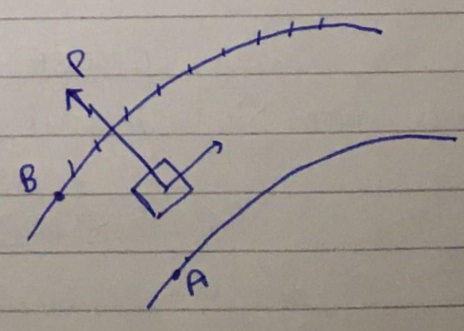
$$\boxed{R_2 = R_1 + \frac{f}{b/2h}}$$

{ f_{max} for above fig. = μmg

When a veh. Negotiate a horizontal curve then the pressure on outer wheel > inner wheel, so long friction is mobilised inwards

Case-(ii) When the vehicle is moving on a banked Road





$N = R_1 + R_2 = mg\cos\theta + P\sin\theta$

A) For NO- overturning.

In the limiting case when veh. is just about to overturn about the outer wheel ($R_1=0$), Balancing Moment about the outer wheels we've

$$P\cos\theta \times h = (mg\cos\theta + P\sin\theta)\times \frac{b}{2} + mg\sin\theta \times h$$

Hence for no overturning we should have

$$P\cos\theta \times h \le (mg\cos\theta + P\sin\theta)\times \frac{b}{2} + mg\sin\theta \times h$$

$$P\cos\theta \times h - P\sin\theta \times \frac{b}{2} \le mg\cos\theta \frac{b}{2} + mg\sin\theta \times h$$

$$P\cos\theta h \left(1 - \frac{b}{2h}\tan\theta\right) \le mg\cos\theta h \left(\frac{b}{2h} + \tan\theta\right)$$

$$\boxed{\frac{v^2}{2g} = \quad \frac{P}{mg} \le \frac{b/2h + \tan\theta}{1 - b/2h \tan\theta}}$$

B) For NO skidding

In the limiting case when vehicle is just about to skid ($f = \mu(N) = \mu(mg\cos\theta + P\sin\theta)$), Balancing Forces we've

$$P\cos\theta = mg\sin\theta + \mu(mg\cos\theta + P\sin\theta)$$

Hence for No skidding we should have

$$P\cos\theta \le mg\sin\theta + \mu(Mg\cos\theta + P\sin\theta)$$

$$P\cos\theta - \mu P\sin\theta \le mg\sin\theta + \mu mg\cos\theta$$

$$P\cos\theta(1 - \mu\tan\theta) \le mg\cos\theta(\mu + \tan\theta)$$

$$\frac{P}{mg} \le \frac{\mu + \tan\theta}{1 - \mu\tan\theta}$$

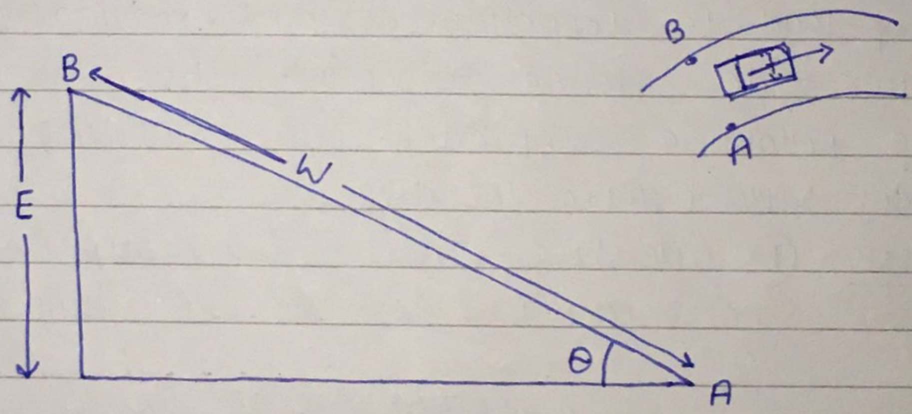
$$\boxed{\frac{v^2}{gR} \le \frac{\mu + \tan\theta}{1 - \mu\tan\theta}}$$

$v \rightarrow$ Design speed (m/s)
$R \rightarrow$ Radius of curve (of center line)
$b \rightarrow$ c/c distance b/w inner & outer wheels
$h \rightarrow$ height of C.G above Road surface
$\mu \rightarrow$ friction cofficient in transverse dirⁿ (0.15)

* Generally skidding takes place before overturning.

SUPERElevation :-

superelevation or bank or cant is the transverse slop provided at horizontal curves to counter act of centrifugal force by raising the outer edge of Road with respect to inner edge.



superelevation (e) = $\tan\theta = \sin\theta = \frac{E}{W}$ ← rise / ← width

for exactly balancing forces in case of NO skidding we've

$$\frac{v^2}{gR} = \frac{\mu + \tan\theta}{1 - \mu\tan\theta} = \frac{\mu + e}{1 - \mu e}.$$

$$\boxed{\frac{v^2}{gR} = \mu + e}$$

$\{\because \mu \times e << 1$ (0.15, 0.07) if friction is not mobilised i.e $\mu = 0$ we've

$$\frac{v^2}{gR} = e$$

such a super-elevation is called equilibirium superelevation

At equilibirium superelevation since friction is not mobilished the pressure on inner and outer wheel is same.

NOTE An adequaly super elevated Road means that equilibirium superelevation has been provided

Guidelines for providing superelevation (For Mixed Traffic)

Equilibirium super elevation provided for design vehicle may lead to toppeling of slow-moving vehicles on the inner side. Hence to counter this IRC has recommended the following approach for providing super elevation under mixed traffic condition.

Steps. ① calculate equilibirium superelevation corresponding to 75% of design speed

$$e = \frac{(0.75v)^2}{gR} = \frac{V^2}{225R} \qquad V \to \text{kmpH}$$

if the calculated $e \leq 0.07$ it is acceptable and if calculated e comes greater than 0.07 then move to Next step.

② provide a superelevation of 0.07 and check for friction cofficient.

$$\frac{v^2}{gR} = \mu + e \quad \left\{ \frac{V^2}{127R} = \mu + e \qquad V \to \text{kmpH} \right.$$

$$\rightarrow \left(\mu = \frac{v^2}{gR} - 0.07 \right)$$

if the calculated $\mu < 0.15$ then acceptable
o/w move to next step

③ calculate the Max. permissable velocity on that Road

$$V_{max.} = \sqrt{(\mu + e)_{max.} gR}$$

(μ → 0.15, e → 0.07)

if the Design velocity is less than $V_{max.}$ → acceptable
if not then restrict the design velocity to ~~to~~ $V_{max.}$

① $\frac{v^2}{gR} = \mu + e$ need / required

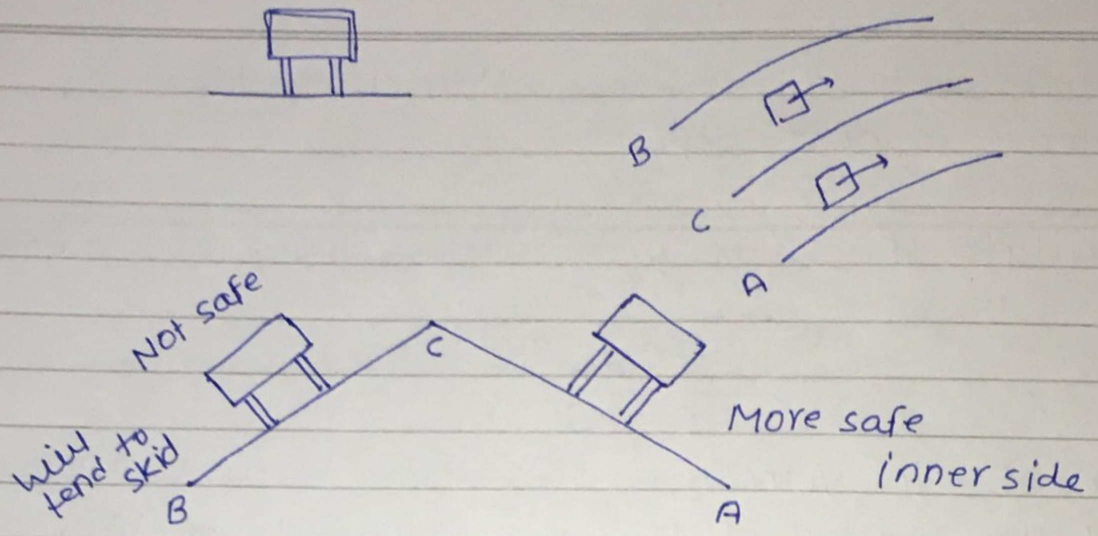
② $\frac{v^2}{gR} = e_{eq.}$ Equivelent superelevation
or adequetly superelevated
or $R_1 = R$

③ $\frac{(0.75v)^2}{gR} = e$ IRC / Design / Mixed traffic

IRC recommendation for ~~Mixed traffic~~ Max. superelevation:–

Type	$e_{max.}$
1) Urban Roads	4 %
2) Plain / Rolling	7 %
3) Hilly terrain bounded by SNOW	7 %
4) Hilly terrain not bounded by SNOW	10 %

NOTE $e_{min.}$ is provided for drainage of water, Hence camber is taken as the value of Min. super elevation



check: $$\frac{v^2}{2g} \leq \mu - c$$ C → slop of camber

When camber is taken as superelevation we must check for the stability of the vehicle on the outer half of the pavement for skidding since the outer half now has a Negative superelevation.

Min. Radius of curve beyond which no superelevation is provided as per IRC is

$$R_{Min} = \frac{(0.75v)^2}{gc}$$

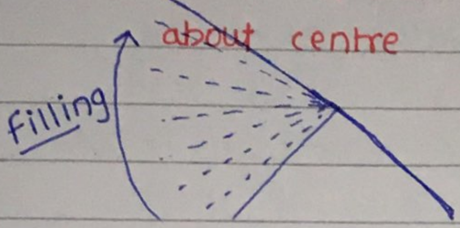
Attainment of superelevation (straight camber)

superelevation is attained in following 2 steps

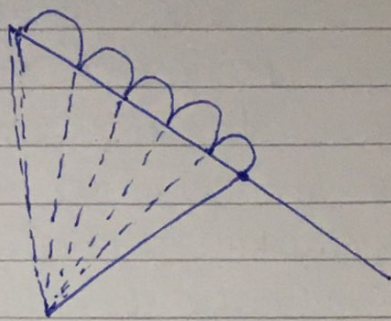
1) elemination of crown
2) Rotation of pavement to archive full superelevation

i) Elimination of crown:-

(i) Rotation of outer pavement about centre



(ii) shifting of crown



Advantage of (i)

- Negative superelevation is never too high in the outer pavement

Dis. (i) Drainage problem at some stage of Road

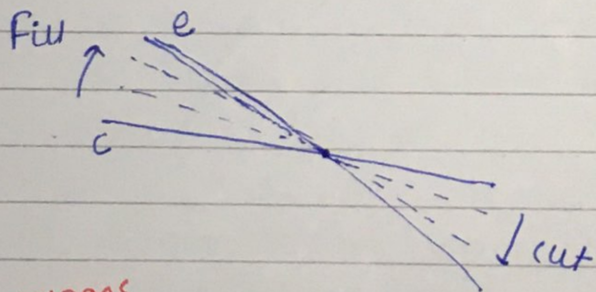
Advantage of (ii)

No Drainage problem anywhere

Dis (ii) super elevation too high in the outer pavement

ii) Rotation of pavement to archive full super elevation

(i) Rotation about centre

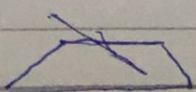


Advantages

- Balance Earthwork is required
- centre line profile of the Road does not changes

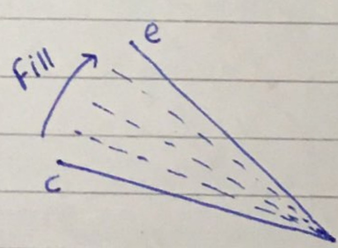
Dis-advantage

- There may be drainage problem on the inner side of the Road



✓ preferred.

(ii) Rotation about inner edge



Advantage.

- No Drainage problem on the inner side

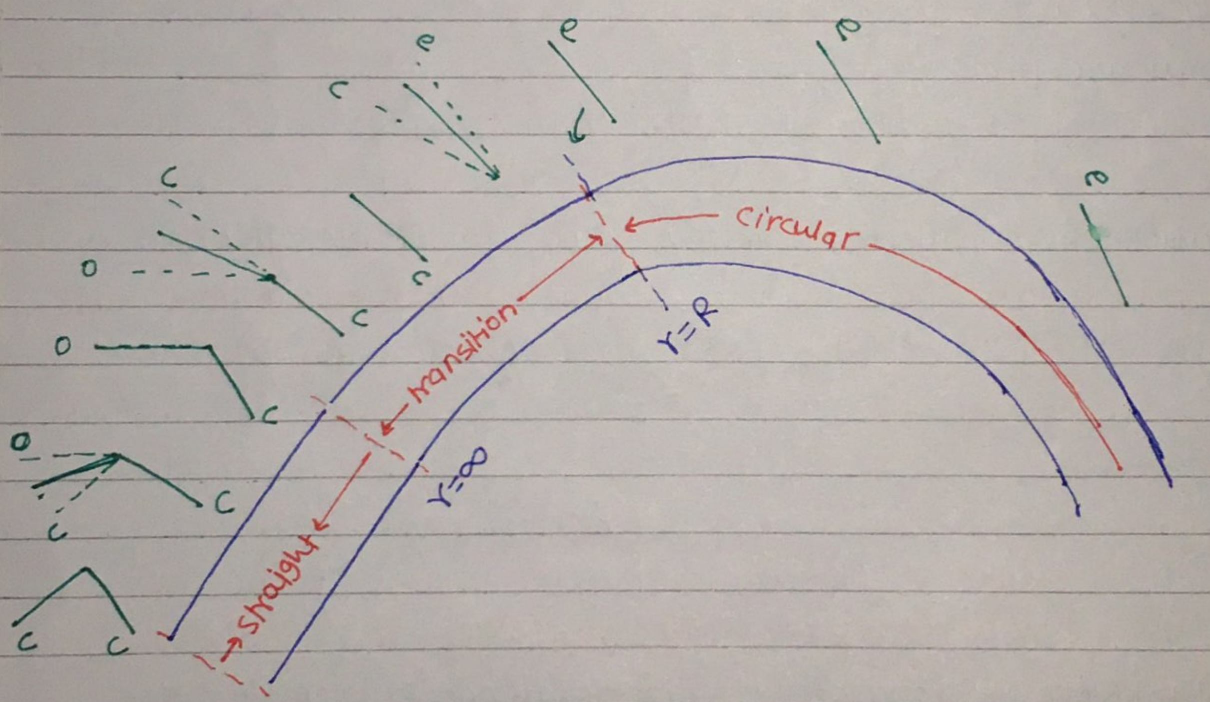
DisAdvantage

- Excess filling earthwork is required
- center line profile of the Road changes.

CROSS-SECTION Along Transition curve :-

We don't start circular curve immediatly after the straight portion of the Road because it will lead to jerk due to sudden introduction of centrifugal force Hence a transition curve is provided b/w the straight and circular portion such that the centrifugal force is Gradually introduced from $0 \rightarrow \frac{mv^2}{R}$

(0: Start of transition curve; $\frac{mv^2}{R}$: end of the transition curve or start of the circular curve)



At the begining of transition curve one leg of the cambered section is made Horizontal and by the end of Transition curve full super elevation is achived

NOTE if Transition curve is not provided then $\frac{2}{3}$ rd of the superelevation is provided on the straight portion of the Road and the remaining $\frac{1}{3}$ rd is provided on the circular portion.

Ruling Minimum Radius and Absolute Min. Radius of Curve:-

$$\frac{v^2}{gR} = \mu + e \Rightarrow R = \frac{v^2}{(\mu+e)g}$$

$$R_{Min.} = \frac{v^2}{(\mu+e)_{Max.}\, g}$$

$$R_{Ruling\ Min.} = \frac{v^2_{Ruling}}{(\mu+e)_{Max.}\, g}$$

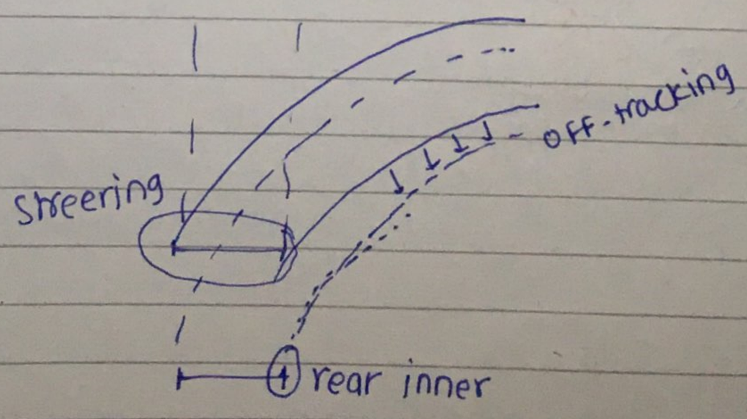
$$R_{Absolute\ Min.} = \frac{v^2_{Min.\ design\ speed}}{(\mu+e)_{max.}\, g}$$

EXTRA-WIDENING ON HORIZONTAL CURVES (W_e):-

Extra widening on Horizontal curves is required for two reasons -

(i) if the front wheel (steering wheel) follows the curve, the inner rear wheel will go off track due to Rigidity of wheel base

(ii) Generally the driver has a tendency to drive closer to the outer edge



For Rigidity OF Wheel Base we provide Mechanical widening $\left(\frac{l^2}{2R}\right)$ per lane

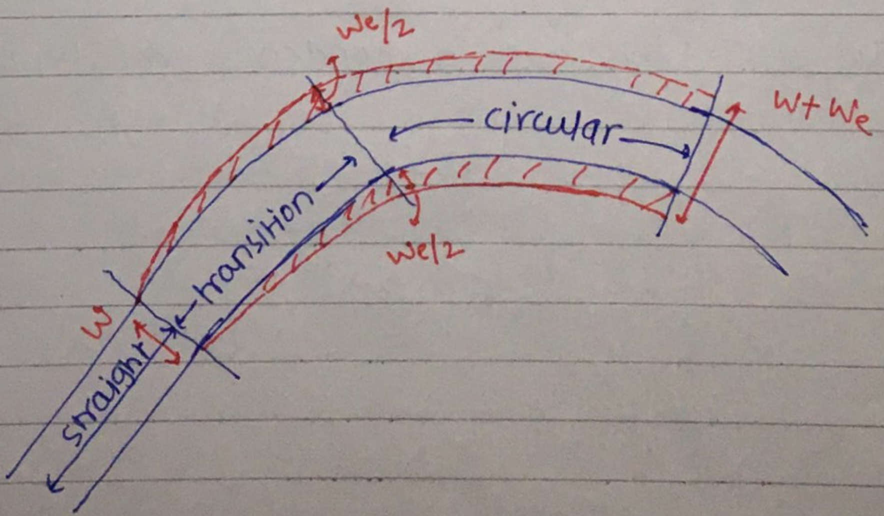
And For tendency of driver to drive closer to the outer edge we provide physcological widening $\left(\frac{v}{2.64\sqrt{R}}\right)$

Total extra widening for n lane

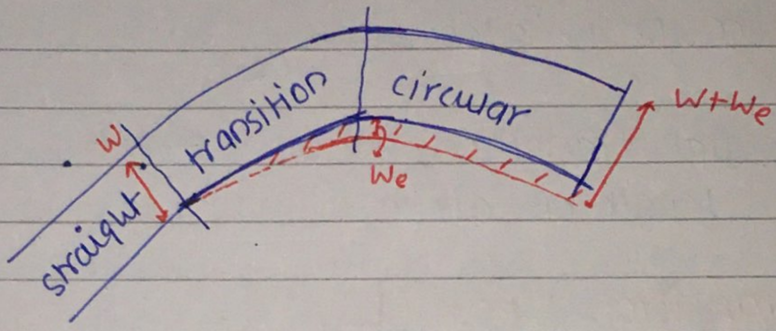
$$W_e = \frac{nl^2}{2R} + \frac{v}{2.64\sqrt{R}}$$

$l \rightarrow$ length of wheel Base
$R \rightarrow$ Radius of curve
$v \rightarrow$ design speed in m/s

- for single lane, only Mechanical widening is provided
- If $R > 300$ m, extrawidening is not required
  Extra widening is provided along the transition curve Gradually



on sharp curv on hills extrawidening is provided on the inner side



If transition curve is not provided then 2/3rd of the extrawidening is provided on the straight portion of the road and the remaining 1/3rd is provided on the circular portion of the Road.

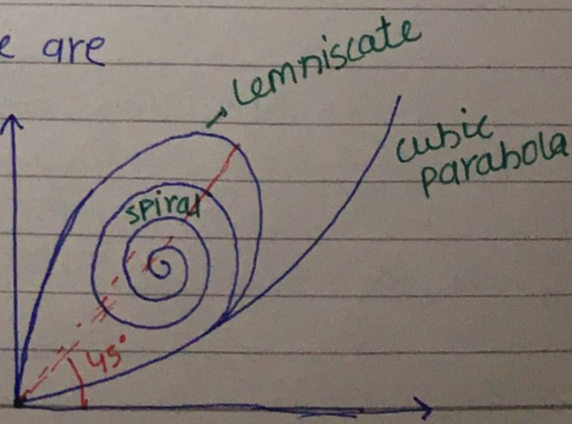
Horizontal Transition curve

objectives of providing horizontal transition curve are

i) Gradual introduction of centrifugal force to facilitate jerk free movement.
ii) Gradual introduction of superelevation and extrawidening
iii) To enable the driver to tern the steering gradually
iv) To enhance the appearance of Road.

Different type of transition curve are

i) spiral (clethoid)
ii) Lemniscate
iii) cubic parabola



IRC Recommends spiral as the shape of Horizontal transition curve.

In case of spiral curve we've $l \propto \frac{1}{R}$, $p \propto \frac{1}{R}$

centrifugal force p is also $p \propto \frac{1}{R}$

Hence for spiral transition we've $p \propto l$.
Where $l \rightarrow$ length of curve

LENGTH OF Transition curve (L_s)

length of Transition curve is taken as Max of length calculated from the following two criterias.

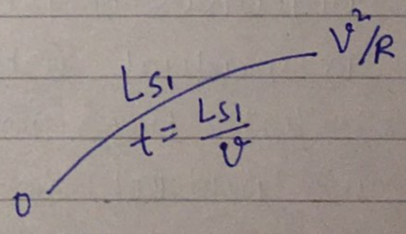
(i) Rate of change of centrifugal acceleration

(ii) Rate of change of introduction of superelevation

(i) Rate of change of centrifugal acceleration :-

Let c be the rate of change of centrifugal acceleration

$$c = \frac{\left(\frac{v^2}{R} - 0\right)}{t} = \frac{\frac{v^2}{R}}{L_{s1}/v}$$



$$c = \frac{v^3}{L_{s1} R}$$

$$\boxed{L_{s1} = \frac{v^3}{cR}}$$

$v \rightarrow m/s$

$c \rightarrow m/s^3$

c is calculated emperically as $\boxed{c = \frac{80}{75 + V_{(KMPH)}}}$

subjected to a Min $0.5\ m/s^3$ to $0.8\ m/s^3$

(ii) Rate of Introduction of superelevation

Rate of change of superelevation that is longitudnal Grade of pavement edge as compared to the through Grade along the center line should be such as not to cause dis-comfert to the traveller or to Make the Road appear unsightly

Rate of change should not be steeper than 1 in 150 For plain / Rolling terrain and should not be steeper than 1 in 60 for Mountaineous / steep terrain.

Based on these statements IRC has given emperical EQ^n to calculate Length of transition curve

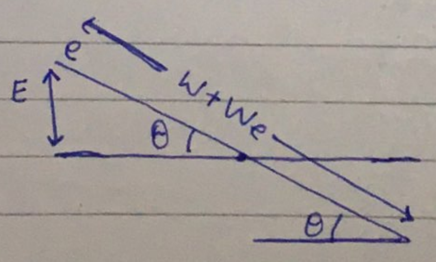
$L_{s_2} = \frac{35 v^2}{R}$	→ plain / Rolling Topography
$L_{s_2} = \frac{12.96\, v^2}{R}$	→ Mountaneous / Steep

$v \rightarrow m/s$

length of transition curve = L_s = Max. $\{L_{s_1}, L_{s_2}\}$

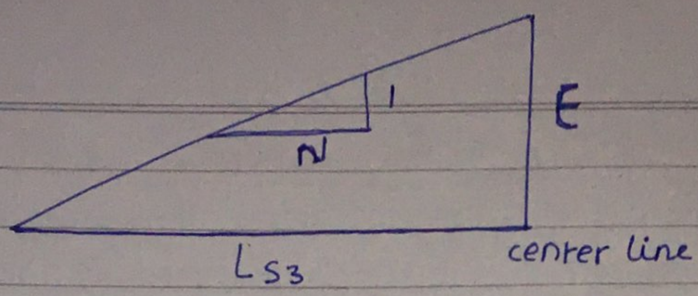
NOTE sometimes length of transition curve is also calculated analytically Based on the 2nd criteria As follow :-

a) When rotation is about center



$e = \tan\theta = \sin\theta = \frac{E}{\frac{(W+W_e)}{2}}$

$E = \frac{e(W+W_e)}{2}$ ---(i)



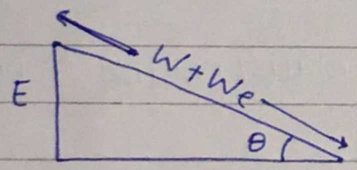
$$\frac{E}{L_{s3}} = \frac{1}{N}$$

$$E = \frac{L_{s3}}{N} \quad \text{----(ii)}$$

from eq^n (i) and (ii)

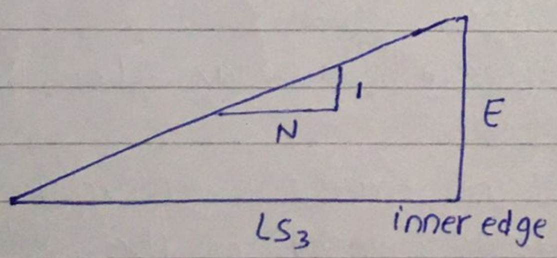
$$\boxed{L_{s_3} = \frac{eN(W+W_e)}{2}}$$

b) When rotation is about inner edge



$$e = \frac{E}{W+W_e}$$

$$E = e(W+W_e) \quad \text{----(i)}$$



$$\frac{E}{L_{s_3}} = \frac{1}{N}$$

$$E = \frac{L_{s_3}}{N} \quad \text{----(ii)}$$

from (i) and (ii)

$$\boxed{L_{s3} = eN(W+W_e)}$$

$e \rightarrow$ superelevation

$W \rightarrow$ width

$W_e \rightarrow$ extra-widening

$N \geq 150$ for Plain/Rolling

$N \geq 60$ for Mountaneous/steep

Length of transition curve $L_s = \max\{L_{s1}, L_{s2}, L_{s3}\}$

Que. An expressway has 4 lanes and it is a devided highway. The expressway is passing through a flat terrain as a horizontal curve of Radius = Ruling Min. Radius, design speed = 120 KMPH calculate
a) Ruling Min Radius
b) superelevation
c) extra-widening
d) length of transition curve

Soln $V = 120$ KMPH, $v = 120 \times 5/18 = 33.33$ m/s

a)
$$R_{\text{ruling Min.}} = \frac{v^2_{\text{Ruling}}}{(\mu + e)_{max}\, g} = \frac{33.33^2}{0.22 \times 9.81} = 514.83 \text{ m.}$$
($\mu \to 0.15$, $e \to 0.07$)

b) Assuming Mixed traffic condition
$$e = \frac{(0.75v)^2}{gR} = \frac{(0.75 \times 33.33)^2}{9.81 \times 514.83} = 0.124$$

$0.124 > 0.07$ (Hence not acceptable), Adopt $e = 0.07$.

Check μ
$$\mu = \frac{v^2}{gR} - e = \frac{33.33^2}{9.81 \times 514.83} - 0.07 = 0.15$$

$0.15 \leq 0.15$ Hence acceptable

c)
$$W_e = \frac{4l^2}{2R} + \frac{v}{2.64\sqrt{R}} \qquad l = 6 \text{ m.}$$
$$W_e = \frac{4 \times (6)^2}{2 \times 514.83} + \frac{33.33}{2.64\sqrt{514.83}} = 0.696 \text{ m.}$$

As $R > 300$, we don't need to provide Extra-widening.

d) $Ls_1 = \frac{v^3}{CR}$ $\qquad C = \frac{80}{75+120} = 0.41$ X

take $C = 0.5$ (Min.)

$= \frac{(33.33)^3}{0.5 \times 514.83}$

$= 143.88\ m$

$Ls_2 = \frac{35v^2}{R} = \frac{35 \times 33.33^2}{514.83} = 75.53\ m$

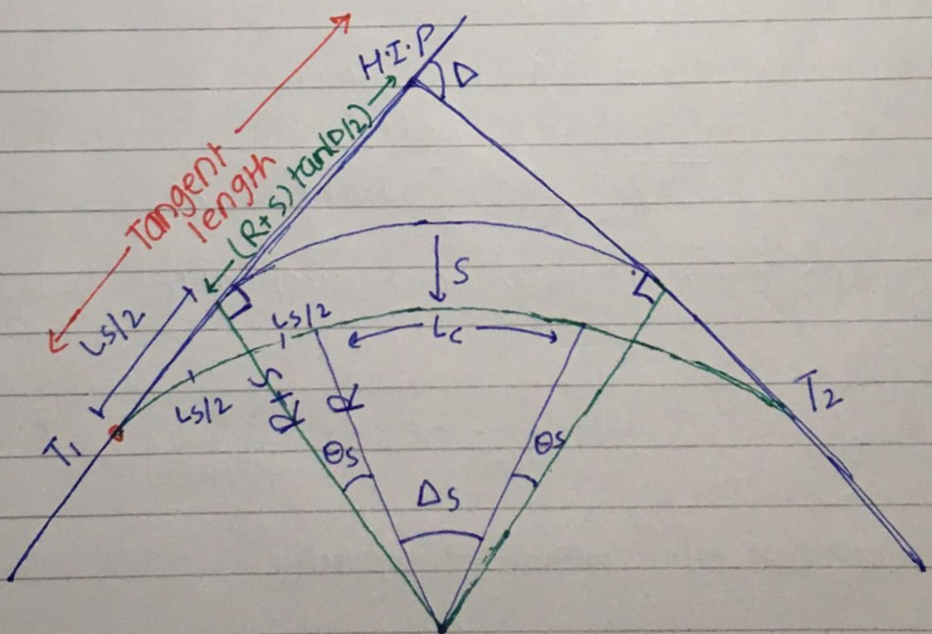
assuming $N=150$

$Ls_3 = \frac{eN(W+We)}{2} = \frac{0.07 \times 150\ (3.5 \times 4)}{2}$

$= 73.5\ m.$

SETTING OF TRANSITION CURVE :-

(spiral & cubic parabola)



$180 - \Delta + 90 + 90 + 2\theta_s + \Delta s = 360$

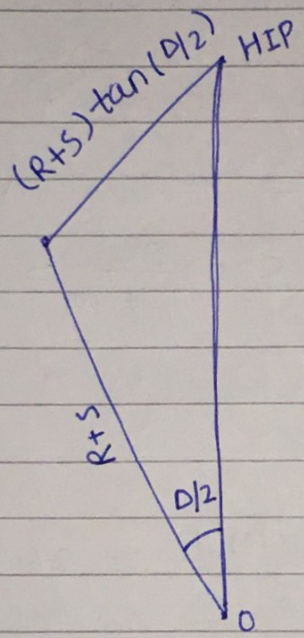
$\boxed{2\theta_s + \Delta_c = \Delta}$

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

$$\theta_s = \frac{L_s}{2R}$$

$$L_c = R\Delta_c$$

Total length of the curve $= 2L_s + L_c$



Tangent length $= \frac{L_s}{2} + (R+S)\tan(D/2)$

- Chainage of first tangent point T_1 = chainage of H.I.P − tangent length
- Chainage of 2nd Tangent point T_2 = Chainage of T_1 + total length of curve

$\Delta \rightarrow$ deflection angle or deviation angle
$\theta_s \rightarrow$ spiral angle
$\Delta_c \rightarrow$ angle subtended by circular portion at center
$R \rightarrow$ Radius of circular curve
$S \rightarrow$ shift
$L_s \rightarrow$ length of transition curve
$L_c \rightarrow$ length of circular curve

NOTE. All angles are in Radian

Que: A two lane pavement 7m wide on a NH on Hilly terrain (SNOW Bound) has a curve of 60m Design speed = 40 KMPH. Determine length of transition curve, total length of curve and tangent length, $\Delta = 60°$. Also calculate the chainage of tangent points if the intersection point has a chainage of 1000m.

$$L_{s_1} = \frac{v^3}{CR} \qquad v = 11.11\ m/s,\ V = 40\ KMPH$$

$$C = \frac{80}{75+40} = 0.695$$

$$L_{s_1} = \frac{(11.11)^3}{0.695 \times 60}$$

$$L_{s_1} = 32.895\ m$$

$$L_{s_2} = \frac{12.96\, v^2}{R} = \frac{12.96 \times (11.11)^2}{60} = 26.66\ m$$

$$L_s = \max.\{L_{s_1}, L_{s_2})$$

$$L_s = 32.895\ m \cong 33\ m$$

length of curve

$$\theta_s = \frac{L_s}{2R} = \frac{33}{2 \times 60} = 0.275\ Rad.$$

$$2\theta_s + \Delta_c = \Delta$$

$$2 \times 0.275 + \Delta_c = \pi/3$$

$$\Delta_c = 0.497\ rad.$$

$$L_c = R\Delta_c = 60 \times 0.497 = 29.831\ m.$$

$$\text{length of curve} = 2L_s + L_c = 2 \times 33 + 29.831 = 95.831\ m.$$

$$\text{Tangent length} = \frac{L_s}{2} + (R+S)\tan \Delta/2$$

$$S = \frac{L_s^2}{24R} = 0.75\,m$$

$$\text{Tangent length} = \frac{33}{2} + (60+0.75)\tan 30^\circ$$

$$= 51.574\,m$$

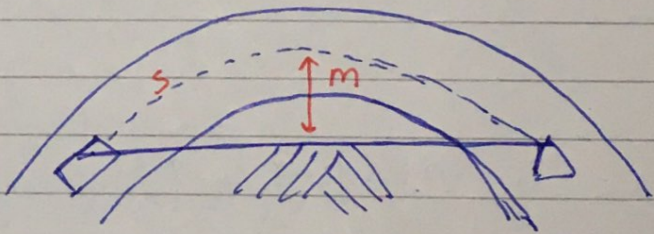
Chainage of 1^{st} tangent point (T_1) = chainage of H.I.P − tangent length

= 1000 − 51.574

= 948.426 m

Chainage of 2^{nd} tangent point (T_2) = 948.426 + 95.831 = 1044.26m

SET BACK Distance :-



set back distance on ~~and~~ clearance distance required from the centerline of horizontal curve -.

to an obstraction on the inner side of the curve is provided so that adequate sight distance is available on the horizontal curve

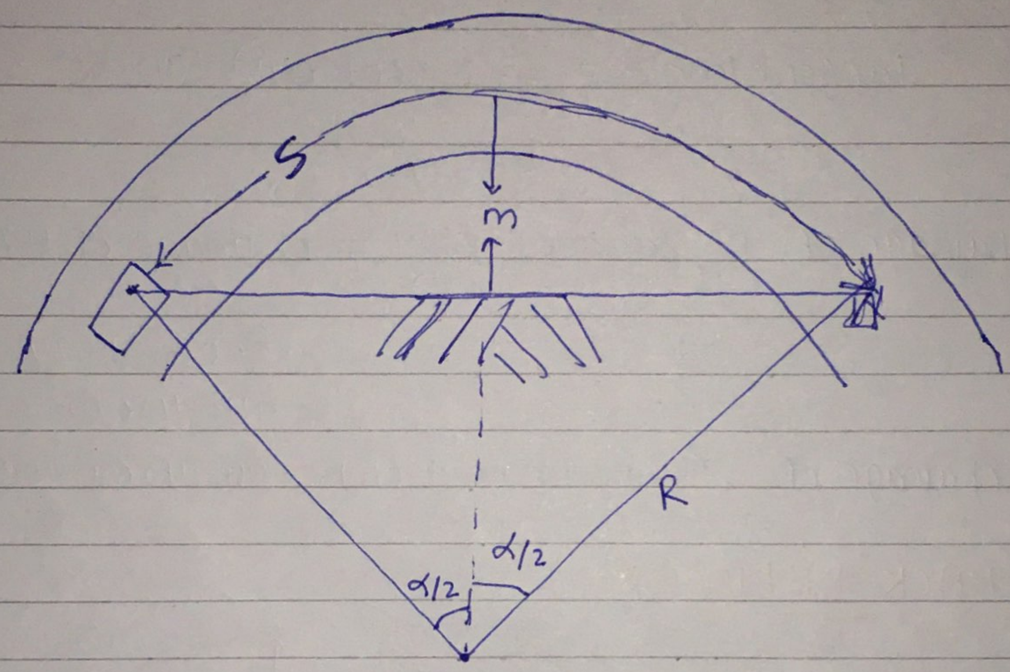
- sight Distance can either be SSD, ISD or OSD
- Min. set back distance should be provided corresponding to SSD

NOTE: In our calculations, we'll calculte the setback distance from the center line of the curve.

case-(ii)

When length of curve > sight distance S

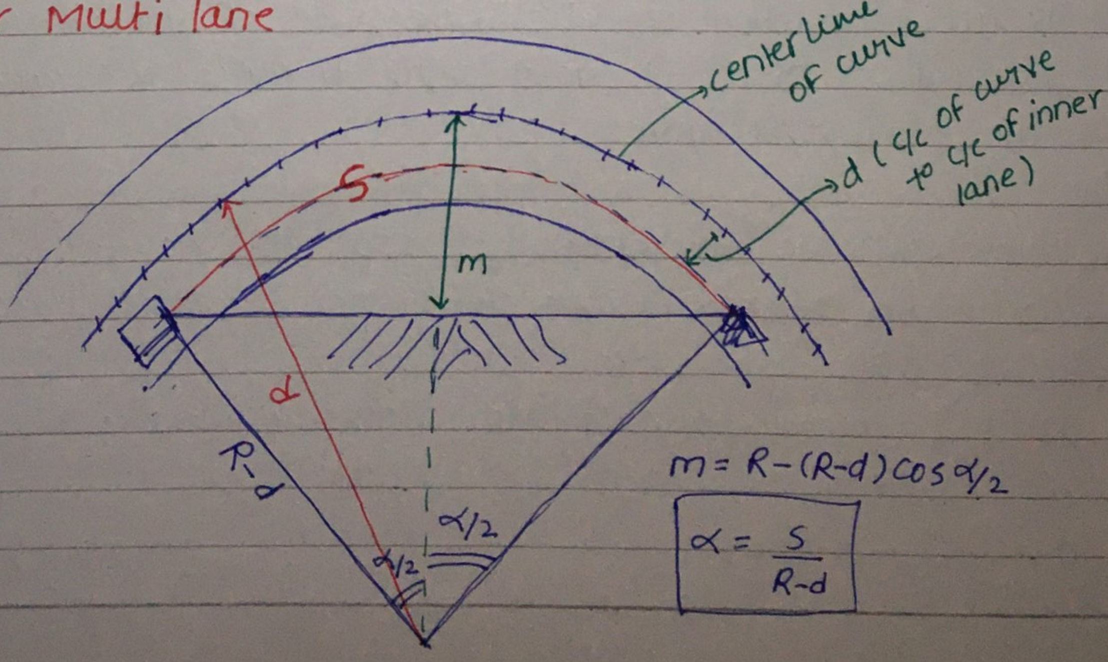
A) For single lane



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

$$\boxed{\alpha = \frac{S}{R}}$$

B) For Multi lane

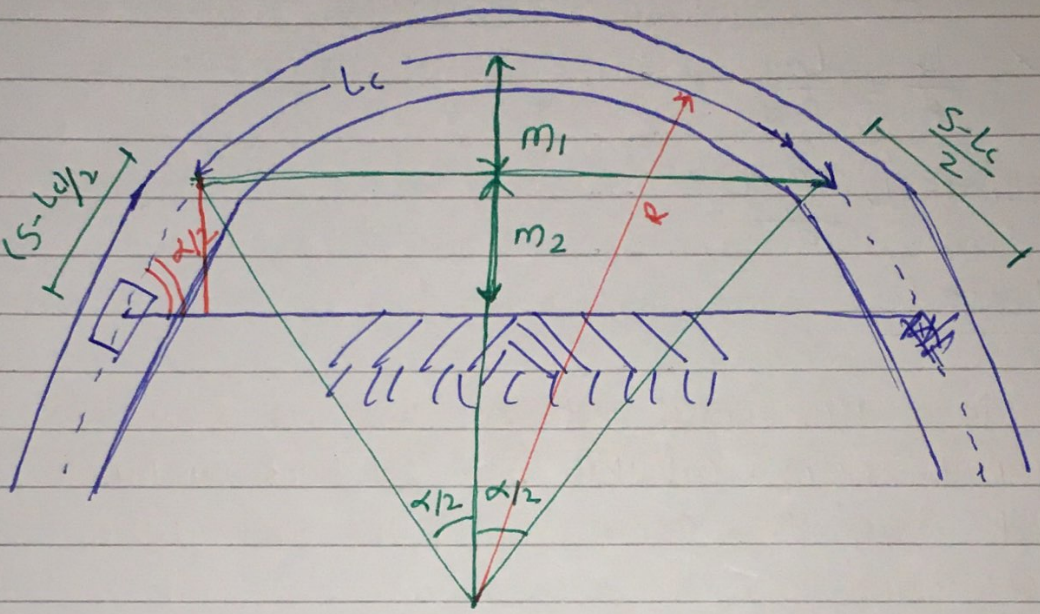


$$m = R - (R-d)\cos\alpha/2$$

$$\boxed{\alpha = \frac{S}{R-d}}$$

case(ii) When length of curve < sight distance S

A) For single lane



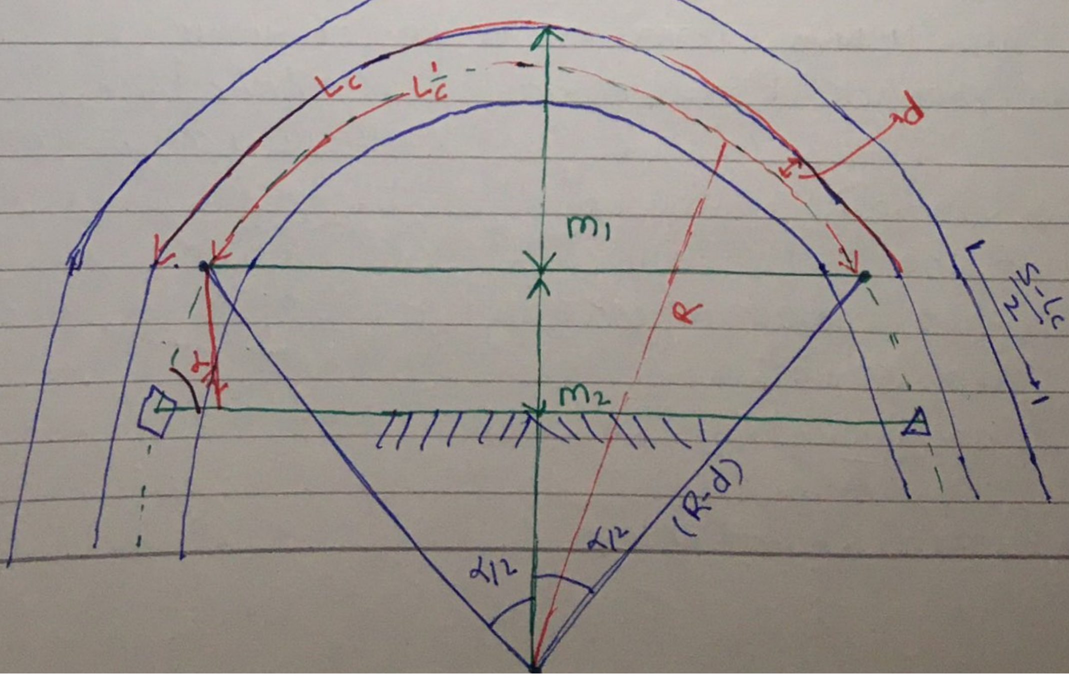
$m = m_1 + m_2$

$m_1 = R - R\cos(\alpha/2)$

$m_2 = \frac{S - L_c}{2} \sin(\alpha/2)$

$\alpha = \frac{L_c}{R}$

B) Multi lane



$m = m_1 + m_2$

$m_1 = R - (R-d)\cos \alpha/2$

$m_2 = \frac{S-L_c}{2} \sin(\alpha/2)$

$\alpha = \frac{L_c}{R} = \frac{L_c'}{R-d}$

$L_c' = L_c \frac{(R-d)}{R}$

$L_c' \simeq L_c$

Que. There is a Horizontal highway curve of R=400 m and length 200 m. Calculate the set back distance required from the center line of the inner lane for providing

a) An SSD of 90 m.

b) An OSD of 300 m.

The distance b/w center line of Road and the center line of inner road is 1.9 m.

a) $\alpha = \frac{S}{R-d} = \frac{90}{400-1.9} = 0.226$ rad

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

$m = 4.44$ m. from center line of curve

Hence setback from center line of inner lane

$= 4.44 - 1.9 = 2.54$ m.

b) $L_c < S$ (OSD)

$L_c = 200$

$R = 400$, $S = 300$ m (OSD)

$m = m_1 + m_2$

$\alpha = \frac{L_c}{R} = 0.5$

$m_1 = R - (R-d)\cos \alpha/2$

Scanned with CamScanner

$m_1 = 400 - (400-1.9)\cos\left(\frac{0.5}{2}\text{ rad.}\right)$

$m_1 = 14.27$ m.

$m_2 = \frac{S-L_c}{2}\sin(\alpha/2)$

$m_2 = \frac{300-200}{2}\sin\left(\frac{0.5}{2}\text{ rad.}\right)$

$m_2 = 12.37$ m

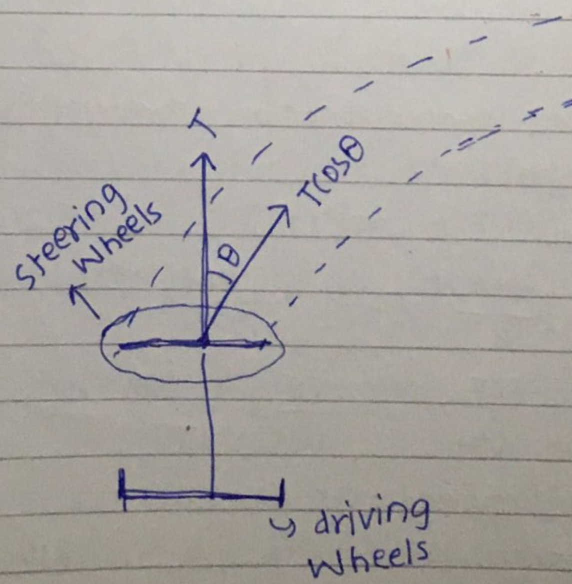
$m = 26.64$ m from C/L of curve

set back distance from C/L of inner lane $= 26.64 - 1.9$

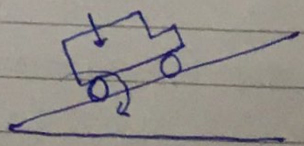
$= 24.74$ m.

CURVE RESISTance and GRade compentation :-

Most of the commercial veh. have rear driving wheels that is thrust is produced by Rear wheels. When such a vehicle is negotiating a horizontal curve the front wheels (steering wheels) are rotated by the driver hence only a component of thrust generated is available to negotiate the curve. This reduction in thrust $(T - T\cos\theta)$ is called Curve Resistance.



curve resistance $= T - T\cos\theta$



NOTE When front wheels are driving wheels (as in passanger cars) there will be no curve resistance

Grade compentation :- When horizontal curve exists together with upgradient the pulling power of veh. decrease. To increase the pulling power the gradient is decreased so as to compentate the loss of tractive effort. This reduction in gradient on horizontal curves is called Grade compentation.

Grade compentation is not required on grades flatter than 4 %

The Grade compentation is calculated emperically as $\frac{30+R}{R}$ % subjected to a Max. of $\frac{75}{R}$ %

Where R → Radius of curve (m)

NOTE. The compentated gradient should not be flatter than 4 %.

Que. if the Ruling Gradient is 1 in 20 what is the Grade compentation and compentated gradient of a curve of Radius 120 m.

Existing Gradient → 1/20 = 5 %

$$\text{Grade compentation} = \frac{30+R}{R}\,\% = \frac{30+120}{120} = 1.25\,\%$$

$$\text{Max. Grade compentation} = \frac{75}{R}\,\% = \frac{75}{120} = 0.625\,\%$$

Grade compentation provided = 0.625 %

Compentated Gradient = 5 − 0.625 = 4.375 %

VERTICAL ALIGNMENT DETAILS :-

Vertical Alignment of Road is decided in such a way that

1) Gradient doesn't become excessive
2) There shouldn't be any drainage problem in the sagging portion of the curve
3) As far as possible cutting should be equal to filling.
4) sufficient sight distance is available at every point of the curve.
5) Aesthetic

Types of Longitudnal Gradient :-

(i) **Ruling Gradient :-** It is the Max. design gradient upto which a designer attempts to design the vertical profile of the Road. It is taken as the Gradient on which, with it's Max. pulling power, a vehicle is able to maintain a constant speed over a long streach.

(ii) **Limiting Gradient :-** It is adopted when Ruling Gradient leads to enormous increase in cost. It will be steeper than Ruling Gradient but the streach Gradient should be limited (as short as possible) and it should be sandwiched b/w flatter Gradients. (or level Grades)

(iii) **Exceptional Gradient :-** It is very steep Gradient but the strech should not exceed 100 m. On both sides of exceptional Gradient there should be milder Gradients for a Minimum length of 100 m.

IRC Recomendations :-

Terrain		Ruling	Limiting	exceptional
Plain/Rolling	→	3.33% (1/30)	5% (1/20)	6.67% (1/15)
Mountaneous/steep	→	5%	6%	7%

for objective. *

* hydraulic Gradient = $\sqrt{L^2+C^2}$

* To avoid erosion slop of camber $\geq$ $\frac{\text{slop of longitudnal gradient}}{2}$

NOTE Minimum longitudnal gradient However is provided for drainage purpose

for example → open soil drains → 1/200

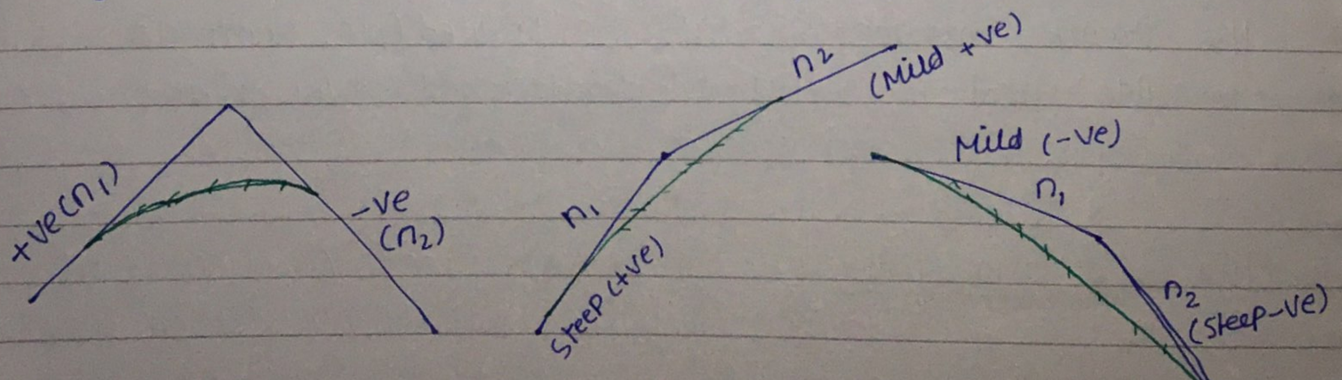
concrete drains → 1/500

Exception → level grades can be provided on fill section.

Critical Length of Grade :- It is the length in which a truck can move without it's speed reducing by more than 25 KmpH.

SUMMIT CURVE / CREST CURVE

It is a curve with convexity upward



$N = (n_1 - n_2)$

In case of summit curve centrifugal force does not creates discomfert hence it is not designed for rate of change of centrifugal acceleration and it only designed for sight distance criteria. From sight distance criteria the ideally suited curve would be circular since the circular curve has equal sight distances from all points. However IRC recommends the use of square parabola for the design of summit curve due to following reasons:-

(i) calculation is easier

ii) Rate of change of grade is constant ⇓

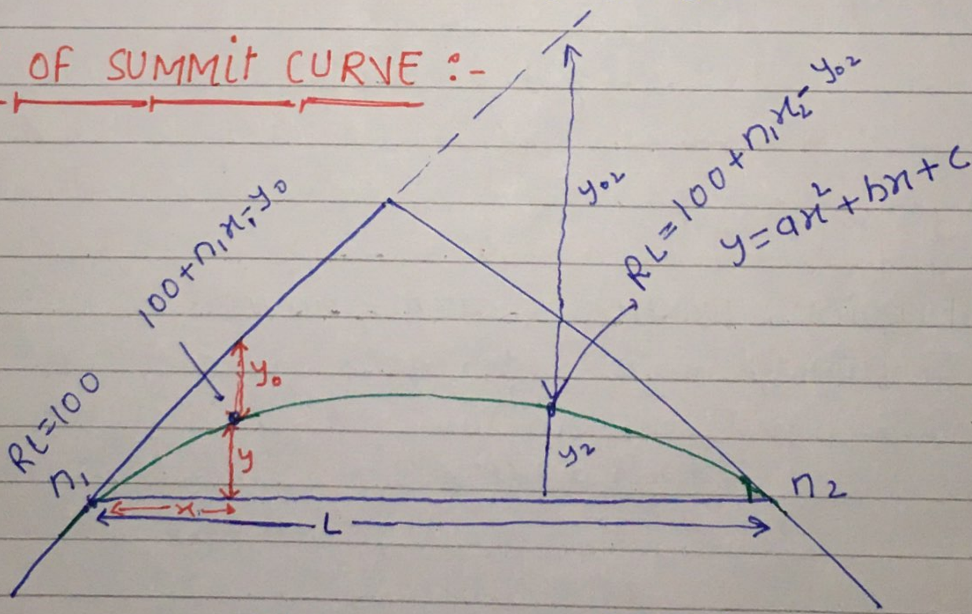
iii) The top portion of parabolic curve is flatter providing greater sight distance.

$$Y = ax^2 + bx + c$$

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

$$\frac{d}{dx}\left(\frac{dy}{dx}\right) = 2a$$

Eq^n of Summit Curve :-



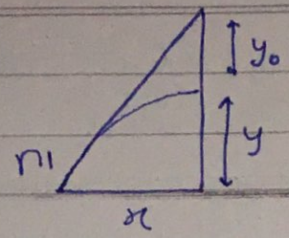
At $x=0, y=0 \Rightarrow \boxed{c=0}$

At $x=0, \frac{dy}{dx} = n_1 \Rightarrow \boxed{b=n_1}$

At $x=L, \frac{dy}{dx} = n_2 \Rightarrow a = \frac{-(n_1-n_2)}{2L}$

$$\boxed{a = \frac{-N}{2L}}$$

$$\boxed{Y = \frac{-N}{2L}x^2 + n_1 x}$$



$$n_1 = \frac{(y_o + y)}{x}$$

$$\boxed{y_o = n_1 x - y}$$

Min. Radius

$$\text{curvature}\left(\frac{1}{R}\right) = \frac{d^2y/dx^2}{\left\{1 + \left(\frac{dy}{dx}\right)^2\right\}^{3/2}}$$

At max. curvature

$$\left(\frac{1}{R_{min}}\right), \quad \frac{dy}{dx} = 0$$

$$\frac{1}{R_{min}} = \frac{d^2y}{dx^2} = 2a$$

$$\frac{1}{R_{min}} = \frac{-N}{L}$$

$$\boxed{|R_{min}| = L/N}$$

Highest point's location on summit curve:-

@ Highest point $\frac{dy}{dx} = 0$

$$2ax + b = 0$$

$$\frac{-N}{L}x + n_1 = 0$$

$$\boxed{x_0 = \frac{n_1 L}{N}}$$ From 1st tangent point

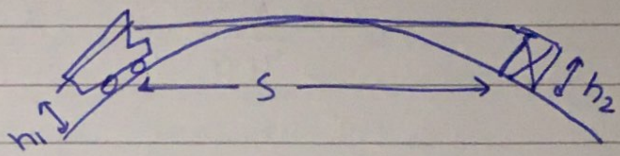
objective

* When the two grades are equal curve will be symmetrical about the verticle bisector and the highest point will lie on this bisector

* in case of unequal grades the highes point will lie on the side of flatter gradient

LENGTH OF CURVE :-

	h_1	h_2
SSD	1.2 m	0.15 m
OSD	1.2 m	1.2 m



case (i) When length of curve L > sight distance S

$$L = \frac{NS^2}{2(\sqrt{h_1}+\sqrt{h_2})^2}$$

case (ii) When length of curve L < sight distance S

$$L = 2S - \frac{2(\sqrt{h_1}+\sqrt{h_2})^2}{N}$$

$$2(\sqrt{h_1}+\sqrt{h_2})^2 = 4.4 \text{ for SSD}$$

$$= 9.6 \text{ for OSD/ISD}$$

NOTE: if the length requirement works out to be -ve it means that there is no sight distance restriction and in that case we simply smooth out the kink by providing a Nominal length.

IRC Recommendation

speed	Max. grade change in % (N) not requiring curve	curve length
80	0.6	50 m
100	0.5	60 m

Que. Calculate the length of summit curve for an SSD of 180 m when the upgradient is 1/200 and the downgradient is also 1/200

Assuming $L > S$

$$L = \frac{NS^2}{4.4} = \frac{\left(\frac{1}{200} - \left(-\frac{1}{200}\right)\right) \times 180^2}{4.4} = 73.63 m$$

$\not> 180$ hence unacceptable

Assume $L < S$

$$L = 2S - \frac{4.4}{N} = 2 \times 180 - \frac{4.4}{\frac{1}{200} - \left(-\frac{1}{200}\right)}$$

$$= -80 m.$$

Hence No restriction of sight so We'll provide length of the curve as 60 m to smooth out the kink

Que. Design a summit curve for a NH for SSD = 180m when $n_1 = 1/50$, $n_2 = -1/30$. set out the curve with a ~~curve~~ chord length of 25 m and also find the reduced level of the highest point on curve when R.L of First tangent point is 100 m.

$$n_1 = 1/50 \qquad n_2 = -1/30$$

$$N = n_1 - n_2 = 4/75$$

assume $L > S$

$$L = \frac{NS^2}{4.4} = \frac{4/75 \times 180^2}{4.4} = 392.73 m > SSD$$

acceptable

location of highest point $= (x_0) = \frac{n_1 L}{N} = \frac{1/50 \times 392.73}{4/75}$

$= 147.28 m$

$$RL\Big|_{x=x_0} = 100 + n_1 x\Big|_{x=x_0} - y_0\Big|_{x=x_0}$$

$$RL\Big|_{x=x_0} = 100 + \frac{1}{50} \times 147.28 - \frac{N}{2L} x^2 \Big|_{x=x_0}$$

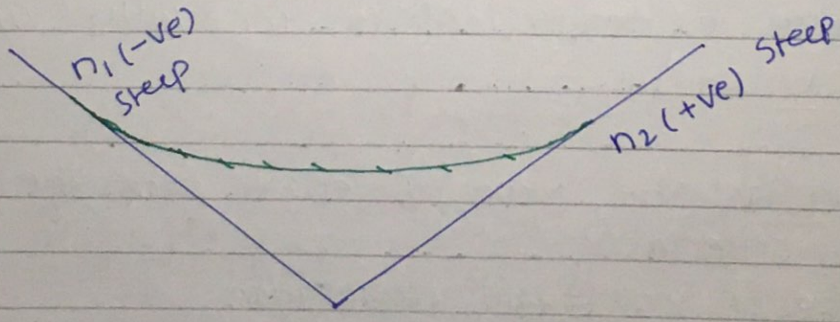
$$= 100 + \frac{147.28}{50} - \frac{4/75 \times 147.28^2}{2 \times 392.73}$$

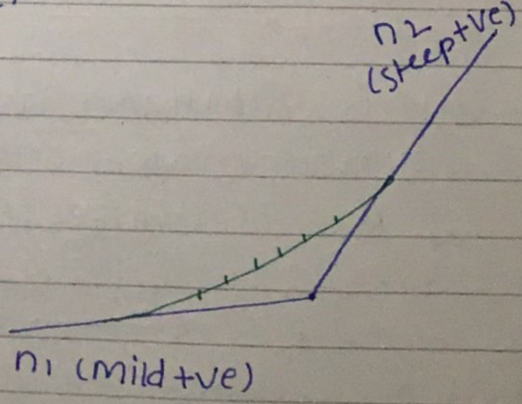
$$= 101.47 \text{ m.}$$

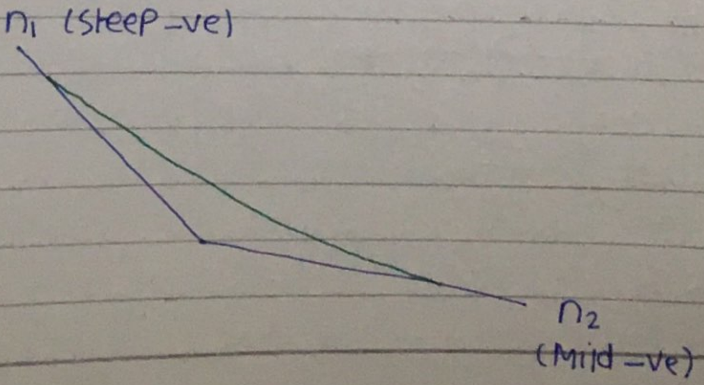
x	$y_0 = \frac{Nx^2}{2L}$	$RL = 100 + n_1 x - Nx^2/2L$
0	0	100
25	0.042	100.458
50	0.164	100.834
⋮	⋮	⋮
392.73	⋮	⋮

VALLEY CURVE or SAG CURVE :-

It is a curve with concavity upward $N = |n_1 - n_2|$







There is no restriction of sight distance on vally curve during day time. However visibility is reduced under No lighting condition at Night. Visibility at Night is only under Headlight sight distance. Hence valley curves are designed for Headlight sight distance (Min value of HSD is taken as SSD)

NOTE There is no problem of overtaking even during Night because of the headlights of the vehicle coming from opposite lane and the rear lights of the vehicle to be overtaken.

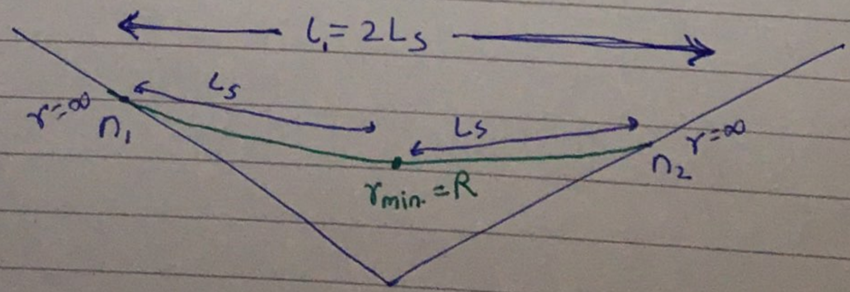
In case of valley curves centrifugal force will be exerted in downward dir^n along with the weight of the vehicle Hence the impact on the vehicle is more. Therefore Rate of change of centrifugal acceleration is also considered in design.

Drainage is also a design criteria for valley curves

LENGTH OF curve :-

It is calculated as the Max. of length from the following two criterias.

i) Rate of change of centrifugal acceleration.



Let c be the rate of change of centrifugal acceleration

$$c = \frac{\frac{v^2}{R} - 0}{t} \Rightarrow c = \frac{v^2/R}{L_s/v} = \frac{v^3}{L_s R}$$

$$c = \frac{v^3}{L_s R} \quad \text{—— (1)}$$

For spiral curves we've $\left\{ R = \frac{L_s}{N} \right.$ —— (2)

from (1) & (2)

$$c = \frac{v^3 N}{L_s^2} \Rightarrow L_s = \sqrt{\frac{N v^3}{c}}$$

Hence length of curve

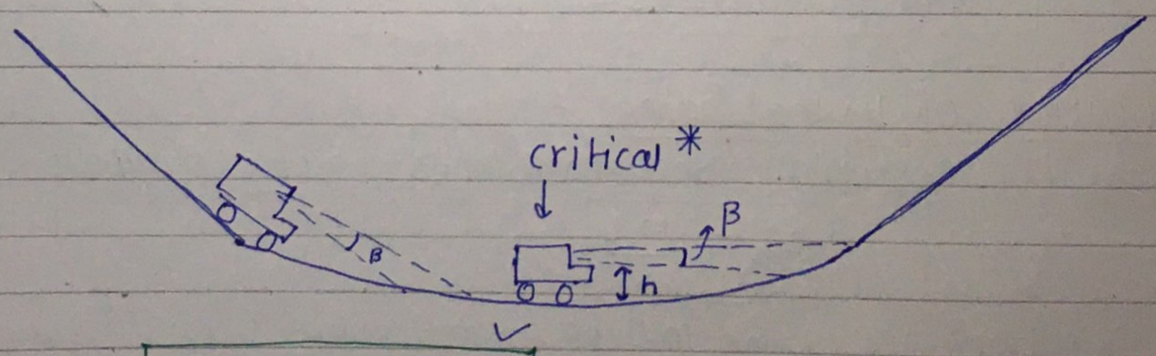
$$L_1 = 2\sqrt{\frac{N v^3}{c}}$$

$c = 0.6\, m/s^3$ (IRC)

(ii) sight distance criteria (S= HSD)

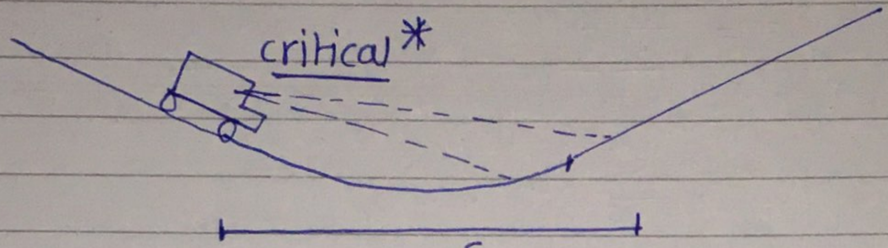
Case(i) When length of curve $L_2 > S$

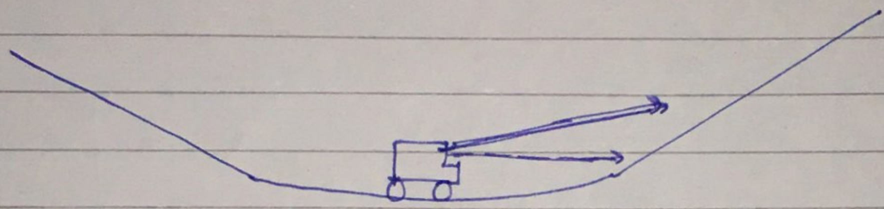
The critical position of veh. is taken at the bottom of the curve!



$$L_2 = \frac{NS^2}{2(h + s\tan\beta)}$$

case-(ii) When length of curve $L_2 < S$





In this case the critical position of veh. is taken at the top of the Road.

$$L_2 = 2S - \frac{2(h + S\tan\beta)}{N}$$

$S \rightarrow$ sight distance (HSD)
$h \rightarrow$ height of headlight above Road surface (0.75m IRC)
$\beta \rightarrow$ Beam Angle (1° as per IRC)

length of curve $L = \text{Max.}(L_1, L_2)$

Location of lowest point :-

case (i) Assuming the curve to be square parabola

$$x_0 = \frac{n_1 L}{N}$$

where x_0 is the locⁿ of lowest point from 1st tangent point
$n_1 \rightarrow$ initial curve

case (ii) Assuming the curve to be cubic parabola

$$x_0 = L\sqrt{\frac{n}{2N}}$$

$x_0 \rightarrow$ locⁿ of lowest point from tangent point on the flatter gradient
$n \rightarrow$ flatter gradient

***

* Valley curve is generally designed as 'cubic parabola'
However IRC recommends 'square parabola'

Que: Valley curve of a straight highway is formed by a descending gradient of 1/20 and ascending gradient of 1/30, Design speed = 80 KMPH. Calculate the length of curve? (Make suitable Assumptions)

$V = 80 \text{ kmph}$

$v = \frac{80 \times 5}{18} = 22.22 \text{ m/s}$

$N = \left| -\frac{1}{20} - \frac{1}{30} \right| = \frac{1}{12}$

$S = HSD \cong SSD = v t_r + \frac{v^2}{2g\mu}$

$= 22.22 \times 2.5 + \frac{(22.22)^2}{2 \times 9.81 \times 0.35}$

$= 127.5 \cong 128 \text{ m}$

Assume $L > S$

$L_2 = \frac{NS^2}{2(h + S\tan\beta)} = \frac{1/12 \times (128)^2}{2(0.75 + 128 \tan 1^\circ)}$

$L_2 = 228.75 \text{ m}$

$L_1 = 2\sqrt{\frac{Nv^3}{C}} = 2\sqrt{\frac{1/12 \times (22.22)^3}{0.6}}$

$L_1 = 78.08 \text{ m}$

Hence $L = \max.(L_1 \& L_2)$

$L = 228.75 \text{ m}$

TRAFFIC - ENGINEERING

The Basic objective of traffic engineering is to achive a free or Rapid flow of traffic with least no. of accidents for this various studies are carried out which are

I) Traffic charecteristic study
II) Traffic Analysis and study
III) Traffic control and regulation

TRAFFIC CHARECTERISTIC STUDY

① Road user charecteristics :-

Mental, physical and physcological study of Road user is carried out

Through these studies PIEV theory (perception, intelection, Emotion, volition) is stabilished which gives the reaction time as 2.5 sec.

② Vehicular charecteristics :-

length, width, height and weight of vehicle is studied

③ Braking charecteristics :-

spacing b/w two consucutive veh. and SSD is affected by Braking charecteristics. To study the braking charecteristics Braking test is performed and skid resistance (longitudnal friction cofficient) is found out atleast two of the following three parameters are required to calculate the value of μ

(a) initial velocity (V)
(b) Braking Distance (l)
(c) Actual Duration of Brake application (t_o)

case (i) When v and l are known

$$v^2 = u^2 + 2as$$
$$0 = v^2 - 2\mu g l$$
$$\boxed{\mu = \frac{v^2}{2gl}}$$

case (ii) When v and t_0 are known

$$v = u + at_0$$
$$0 = v - \mu g t_0$$
$$\boxed{\mu = \frac{v}{g t_0}}$$

case (iii) When l and t_0 are known

$$v = u + at$$
$$0 = v - \mu g t_0$$
$$v = \mu g t_0 \quad \text{— (i)}$$
$$s = ut + \tfrac{1}{2}at^2$$
$$l = v t_0 - \frac{\mu g t_0^2}{2} \quad \text{— (ii)}$$

from (i) and (ii)

$$l = \mu g t_0^2 - \frac{\mu g t_0^2}{2} = \frac{\mu g t_0^2}{2}$$

$$\boxed{\mu = \frac{2l}{g t_0^2}}$$

* Braking efficiency is expressed as $= \dfrac{\mu_{\text{Braking test}}}{\mu_{\text{max. known}}} \times 100$

TRAFFIC ANALYSIS AND STUDY

It helps in analysis the need for Geometrical design features and also intaking traffic control measures.
Various studies carried out in this segment are
1) Traffic volume study
2) Traffic ~~study~~ speed study
3) origin and destination study
4) Traffic flow charecteristic study → In india we follow keep to left or Right hand drive traffic regulation
5) Traffic capacity study
6) parking study
7) Accident analysis

① Traffic volume study :-

Traffic volume or flow is the no. of veh. crossing a point or sec^n on a Road in unit time. It is expressed in veh/hr or pcu/hr

- It is used to measure the quantity of traffic flow.

Complete traffic volume study includes
a) classified volume study :
No. of diff^n type of veh. are counted
b) Directional study :
Distribution of traffic in diff^r lanes is counted
c) Turning Movement study at intersections
It is done for intersection design
d) pedestrian volume study
This helps in planning subways, footbridge and pedestrain signal

presentation of traffic Volume study Data

(i) AADT : (Avg. Annual Daily traffic)

It is the avg. 24 hr vol. at a locn calculated over 365 days

It includes seasonal variation

(ii) ADT : (Avg. Daily Traffic)

Min. 7 day count is taken

it includes weekly variation

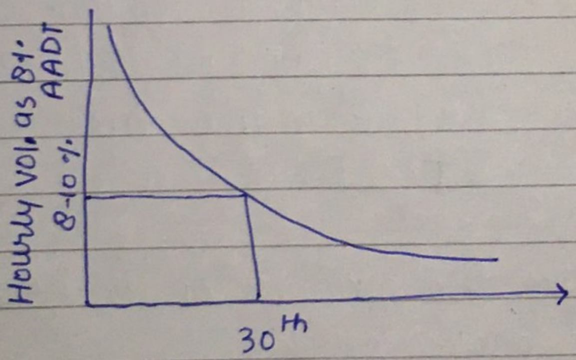
(iii) Trend chart :

Vol. trend over a period of year is calculated

It helps in planning expansions and regulations

(iv) 30th highest hourly volume : 30 HV

It is taken as the design capacity or Design hourly volume. It is exceeded only 29 times in a year



(v) Traffic Flow Map along the route :

Thickness of line represents traffic volume.

It gives an idea of traffic flow at a glance

Calculation of AADT :-

since it is impracticle to calculate traffic volm throughout the year. We calculate AADT using periodic count and preestablished expansion factors

1) hourly expansion factor

$$HEF = \frac{24\ hr\ vol^m}{1\ hr\ vol^m\ of\ a\ perticular\ hour}$$

2) Daily expansion factor:

$$DEF = \frac{\text{Weekly Vol}^m}{\text{Avg. 24 hr. Vol}^m \text{ of a perticular Day}}$$

$$ADT = \frac{\text{Weekly vol}^m}{7}$$

3) Monthly expansion factor:

$$MEF = \frac{AADT}{(ADT)_{month}}$$

Que. A traffic engineer urgently needs to determine AADT on a rural primary road. He collected following Data on a Tuesday in the month of May, DEF & MEF are 7.727 & 1.394 respectively. Calculate the AADT?

time	vehicles	HEF
7-8 am	400	29
8-9 am	535	22.5
9-10 am	650	18.8
10-11 am	710	17.1
11-12 am	650	18.52

$$\text{Avg. 24hr vol}^m \text{ of Tuesday} = \frac{400 \times 29 + 535 \times 22.5 + 650 \times 18.8 + 710 \times 17.1 + 650 \times 18.52}{5}$$

$= 12007.3$ Veh/day

$$\text{Daily expansion factor} = \frac{\text{Weekly Vol}^m}{\text{Avg. 24 hr Vol}^m}$$

Weekly vol^m = $12007.3 \times 7.727 = 92780.40$ veh/week

$$ADT = \frac{\text{Weekly Vol}^m}{7} = 13254.34 \text{ veh/day}$$

$$MEF = \frac{AADT}{(ADT)_{month}}$$

$\Rightarrow AADT = 13254.34 \times 1.394$

$= 18476.54$ Veh/day

$\cong 18477$ veh/day

② Traffic SPEED STUDY :- Speed of diffⁿ veh. varies with respect to time and space

Hence to represent these variations various speeds are defigned —

a) Spot speed : It is the instantaneous speed of a veh. at a perticular location. spot speed is used to design Horizontal and verticle curves, traffic signals, Accident analysis and helps in deciding size of Traffic sign. It is measured using Enoscope, pressure contact tubes loop deflector and dopler radar

b) Average speed : It is the avg. of spot speed of vehicles. It is classified as:

** (i) Time Mean Speed (TMS) :- (V_t) It is the avg of spot speed of vehicles crossing a perticular location in a given interval of time. It is the airthemetic Mean of spot speed of vehicles crossing a perticular location in a given interval of time

$$V_t = \frac{\sum_{i=1}^{n} V_i}{n}$$

$n \rightarrow$ no. of vehicles crossing a perticular location in a given interval of time

$V_i \rightarrow$ spot speed of i^{th} vehicle at that location

(ii) Space Mean speed (SMS). (V_s) :- It is the avg of spot speed of vehicles taken over a certain strech in a perticular instanse of time. It is obtained by deviding total distance travelled by total time taken. simply put it is the harmonic Mean of spot speed of vehicles

$$V_s = \frac{n}{\sum_{i=1}^{n} \frac{1}{V_i}}$$

Scanned with CamScanner

$V_i \rightarrow$ spot speed of i^{th} vehicle

Reciprocal of SMS gives the avg. travel time (per km)

NOTE ① since Airthemetic Mean ≥ Harmonic Mean

*** Hence TMS ≥ SMS

② since SMS gives more weitage to lower velocities Hence it is preferred over TMS in traffic analysis

Que. Result of speed study is given in the form of frequency distribution below :- calculate TMS & SMS ?

	speed range (km/h)	No. of vehicles
3.5	2-5	1
8.5	8-9	4
11.5	10-13	0
15.5	14-17	7

$$V_t = \frac{\sum V_i}{n} = 12.167 \text{ km/H}$$

$$V_s = \frac{12}{1 \times \frac{1}{3.5} + 4 \times \frac{1}{8.5} + 0 \times \frac{1}{11.5} + 7 \times \frac{1}{15.5}} = 9.934 \text{ km/h}$$

c) Running speed :- $\frac{\text{journey length}}{\text{Running time}}$ Delay not considered

- This speed excludes Delays
- It is used to analyse Road conditions

d) Journey speed :- $\frac{\text{Journey length}}{\text{Journey time}}$ Delay considered

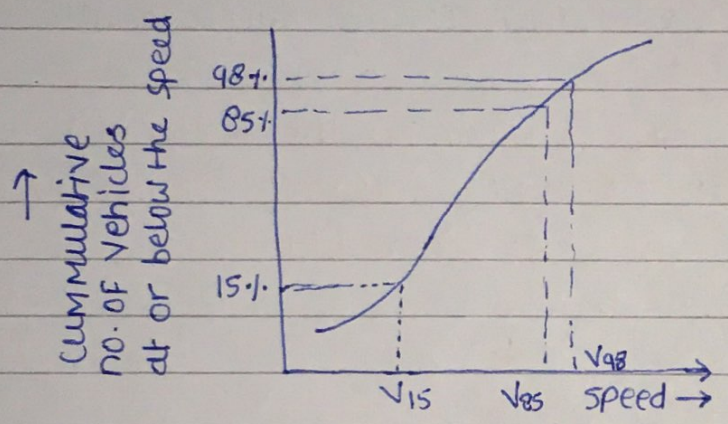
- It includes stop delays
- It is used to analyse traffic flow condition.
- Running speed ≥ Journey speed

Various type of speed studies are -

1) **spot speed study :-** It is used in planning traffic control and traffic regulations. It is represented in the form of -

a) Avg. speed of vehicles - TMS and SMS

b) cummulative speed distribution diagram -



V_{98} percentile speed → It is the speed at or below which 98% vehicles are moving, it is also known as design speed.

V_{85} percentile speed → It is the speed at or below which 85% vehicles are moving. it is known as safe speed or upper limit of speed.

It is used in speed restrictions and some times for calculating OSD

V_{15} percentile speed → It is the velocity at or below which 15% of vehicles are moving. it is taken as the lower limit of speed and attempts are made to segrigate the traffic moving at speeds lower than V_{15} to avoid congestion.

Que. spot speed study is carried out on a strech of Road the data given below. Calculate lower limit of speed, upper limit and design speed.

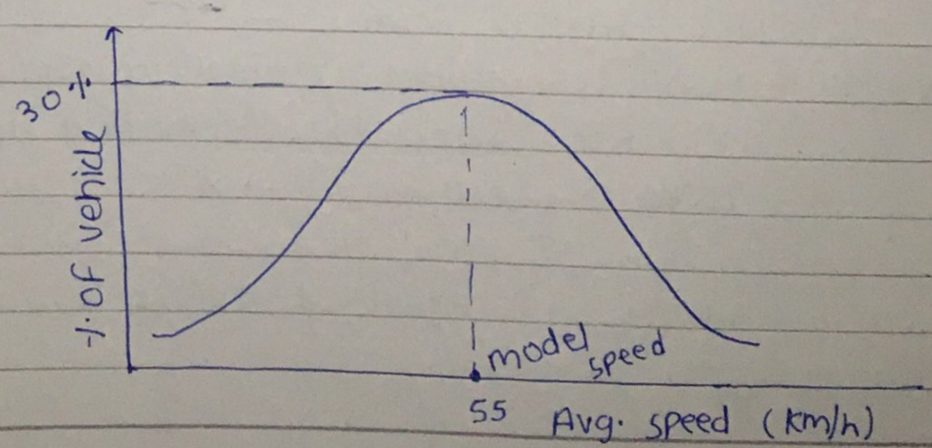
	speed Range (km/hr)	No. of vehicles	% of vehicles	cummulative %
5	0-10	12	$\frac{12}{850} \times 100 = 1.41$	1.41
15	10-20	18	2.11	3.52
25	20-30	68	8	11.52
	V_{15}			15%
35	30-40	89	10.48	22
45	40-50	204	24	46
55	50-60	255	30	76
	V_{85}			85%
65	60-70	119	14	90
75	70-80	43	5.06	95.06
	V_{98}			98%
85	80-90	33	3.88	98.94
95	90-100	9	1.06	100
		$\Sigma = 850$		

$$V_{15} = 25 + \frac{35-25}{22-11.52} \times (15-11.52) = 28.32 \text{ KMPH}$$

$$V_{85} = 55 + \frac{65-55}{90-76} \times (85-76) = 61.42 \text{ KMPH}$$

$$V_{98} = 75 + \frac{85-75}{98.94-95.06} \times (98-95.06) = 82.57 \text{ KMPH}$$

c) Frequency Distribution Diagram — Spot speed study data can be used to determine the speed at which largest percent of vehicles are moving. This speed is called Model speed. It is obtained by plotting the avg. of each speed interval against the % of vehicle in that speed interval.



2) **Speed and delay study** :- These studies are useful in identifying location of congestion, causes and in deriving suitable improvement measures to reduce delay and increase travel speed. These studies are also utilised to find the travel time before and after the proposed improvements. ~~for this~~

· for this analysis is done over a long strech of Road, hence it is also possible to determine traffic density (Vh/km) and flow charecteristic of traffic

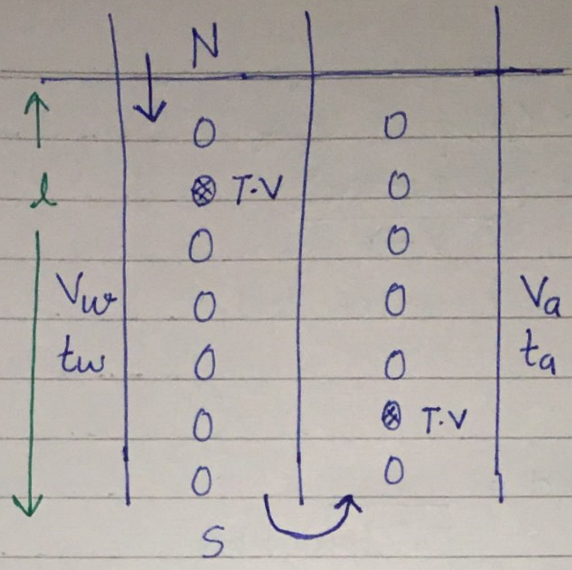
Various Methods of carrying these studies are –

a) Floating car Method
b) License plate Method
c) Interview technique
d) photographic technique

A) **Floating car Method** :- It is suitable for two lane traffic and 4 observers are used:

observer1 → Time at control points and amount of delay
observer 2 → Time, location and cause of delay
observer-3 → No. of Vehicles overtaking the test vehicle (a) and No. of vehicles overtaken by the test vehicle (b), When the test vehicle is moving with the concerned traffic (that is from N → S)
observer-4 → No. of vehicles counted in opposite dirⁿ (x) When the test vehicle is moving against the concerned traffic (S → N)

for our Analysis the concerned traffic is traffic moving from N → S



Y = No. of vehicles overtaking the test vehicle — No. of vehicles overtaken by the test vehicle when the test vehicle is moving with the traffic

Y = a − b (Net traffic veh. overtaking T.V.)

X = No. of vehicles counted in opposite dir^n (N→S) when the T.V is moving S→N

t_w, t_a → journey ~~speed~~ Time of T.V in hour when moving with and against the concerned traffic

V_w, V_a → journey speed of T.V in KMPH when moving with and against the concerned traffic

l → observation length in KM

$\bar{t}$ → avg. journey time in hours for traffic moving from N→S

$\bar{V}$ → avg. speed of traffic in KMPH (N→S)

q → traffic flow in veh/Hr of concerned traffic (N→S)

K → traffic Density in veh/km (N→S)

$$q = \frac{X + Y}{t_w + t_a}$$

$$\boxed{\bar{t} = t_w - Y/q}$$

$$= t_w - \frac{Y(t_w + t_a)}{X+Y}$$

Running time = $\bar{t}$ − stop delays

$$\bar{V} = \frac{l}{\bar{t}}$$

Running speed = $\frac{l}{\bar{t} - \text{stop delay}}$

$$Y = (\bar{V} - V_w) t_w K$$

$$Y = \bar{V} K t_w - V_w t_w K$$

$$\boxed{Y = q t_w - V_w t_w K \quad \text{—(i)}} \quad (q = kv)$$

$$X = (\bar{V} - V_a) t_a K$$

$$\boxed{X = q t_a - V_a t_a K \quad \text{—(ii)}}$$

① + ② $\qquad q = \frac{X+Y}{t_w + t_a}$

From eq^n ①

$$Y = q t_w - V_w t_w K = q t_w - V_w t_w \frac{q}{\bar{V}}$$

$$\frac{Y}{q} = t_w - l/\bar{V}$$

$$(Y/q) = t_w - \bar{t}$$

$$\boxed{\bar{t} = t_w - Y/q}$$

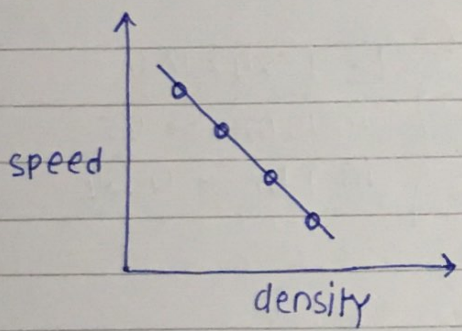
Que. Observation length is 0.5 km and the speed of Test vehicle is 20 kmpH. For the Data given below calculate Avg. speed of traffic stream, traffic Flow & Traffic density. Also plot speed vs Density Relation, speed vs Flow and Flow vs Density Relation.

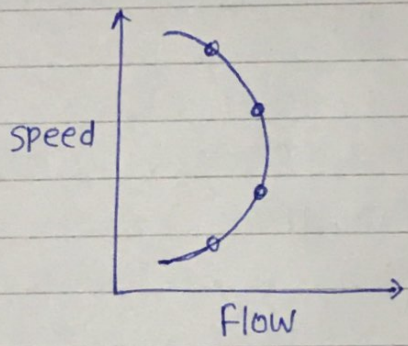
Sample	X	a	b	$Y=a-b$	$q=\frac{X+Y}{t_w+t_a}$ (veh/hr)	$\bar{t}=t_w-Y/q$ (hr)	$\bar{V}=l/\bar{t}$ (km/h)	$K=q/v$ (veh/km)
1	107	10	74	-64	860	0.099	5.05	170.29
2	113	25	41	-16	1940	0.033	15.15	128.052
3	30	15	5	10	800	0.0125	40	20
4	79	18	9	9	1760	0.0198	25.25	69.70

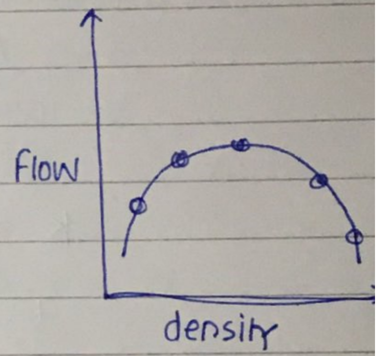
$l = 0.5$ km

$V_w = V_a = 20$ km/hr

$t_w = t_a = \frac{0.5}{20} = 0.025$ hr.







Que. While travelling along the stream and against the stream a moving observer measured the relative flow as 50 veh/hr and 200 veh/hr respectively. The avg. speed of moving observer while moving with or against the traffic is 20 & 30 KMPH respectively. The Traffic density in veh/km?

$$Y = q t_w - V_w t_w K$$

$$\frac{Y}{t_w} = q - V_w K = 50$$

similarly $X = q t_a + V_a t_a K$

$$\frac{X}{t_a} = q + V_a K = 200$$

$$50 = q - V_w K \quad \text{—(i)}$$

$$200 = q + V_a K \quad \text{—(ii)}$$

$$K = \frac{150}{20+30} = 3 \text{ veh/km}$$

Que. A student riding a bicycle on a 5m one way street takes 40 Min to reach home in which he stopped for 15 Min. 45 veh. overtook the student while he stop and 60 veh overtook while cycling. Assuming No. of vehicles overtaken by student to be zero. Calculate the speed of traffic stream on that Road.

$$Y = q t_w - V_w t_w k$$

$t_w = \frac{40}{60}$ hr. $V_w t_w = 5$ km $Y = 45 + 60 = 105$

$q = 45 \times 4 = 180$
in 15 min → 45
in 1 hr → 45×4

$$105 = q \times \frac{40}{60} - 5k$$

$k = 3$ veh/km

$q = kV$

$180 = 3V \Rightarrow V = 60$ km/h Ans

③ origin and DESTINation studies (OD) study :-

OD determines info. like actual duration of travel, selection of route and length of route. These studies helps in planning New highways and in improving existing services. It is also used in designing mass transit systems (Bus, metro)

Various Methods of collecting OD Datas are -

i) Road side interview Method
ii) license plate Method (veh. No. Method)
iii) Return post card Method
iv) Tag on car Method
v) Home interview Method
vi) Work spot interview Method

Objectives

* OD Datas are represented in the form of
  a) Desire lines → thickness of line represents no. of trips
  b) π - charts → Dia of circle represents traffic volume
  c) contoor lines

⑤ TRAFFIC CAPASITY STUDY :-

Traffic volume :- (q)

It is the no. of veh. crossing a given point or section in unit time. It is expressed in Veh/Hr or pcu/hr

** PCU = Capacity with passanger cars only / capacity with corresponding veh. only

$$PCU = \frac{\text{Capacity with passanger cars only}}{\text{capacity with corresponding veh. only}}$$

for ex: pcu for pedal cycle, Moterbike, scooter = 0.5
*** pcu for passenger car, Van, Auto rikshaw = 1
pcu for cycle Rikshaw = 1.5 or 2
pcu for Bus, truck = 3 (congestion & speed)

Traffic Density (k):-

It is the no. of veh. occupying unit length of Road at a given time. It is expressed in Veh/km

Time taken by the last veh. to cross secⁿ ① = 1/v hr.
in $\frac{1}{v}$ hr. no. of veh. crossing secⁿ ① = K
so no of veh. crossing secⁿ ① per hour = $\frac{K}{1/v}$

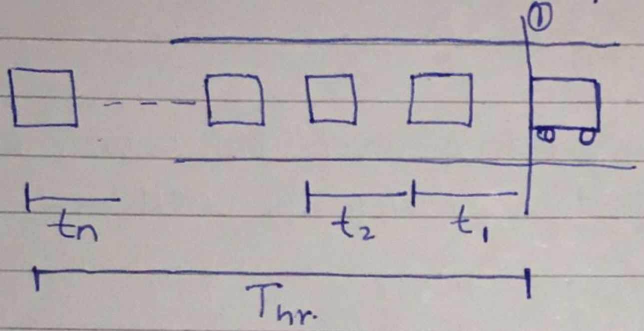
i.e ***$q = KV$***

Where q → traffic vol. in veh./Hr
K → traffic density in veh/km
V → avg. traffic speed in km/hr.

- Highest traffic density will occure when veh. are practically stand still on a given Road or flow = 0

Time Headway

Time interval b/w the passes of rear Bumpers of successive vehicle is called Time headway



let the observation time be T hr.

$$T = \sum_{i=1}^{n} t_i$$

NO. of Vehicles crossing secn ① in T hours = n

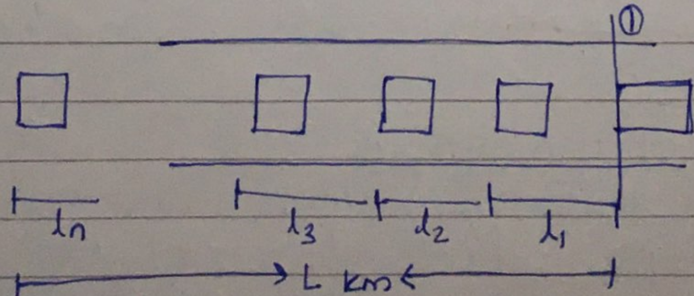
Hence no. of vehicles crossing secn ① per hour (q) = $\frac{n}{T}$

$$q = \frac{n}{T} = \frac{n}{\sum_{i=1}^{n} t_i} = \frac{1}{\left(\frac{\sum_{i=1}^{n} h_i}{n}\right)}$$

$$q_{\text{veh/hr}} = \frac{1}{\text{Avg. time headway}}$$

Space headway

It is the distance b/w rear Bumpers of successive Vehicles



let the observation length be L km.

$$L = \sum_{i=1}^{n} l_i$$

No. of veh. in L km = n

Hence No. of veh. per km (K) = n/L

$$K = \frac{n}{L} = \frac{n}{\sum_{i=1}^{n} l_i} = \frac{1}{\left(\frac{\sum_{i=1}^{n} l_i}{n}\right)}$$

$$K = \frac{1}{\text{Avg. Space headway}}$$

K → veh/km

Traffic capacity

It is the ability of Road to accomodate Max. traffic Vol^m at a perticular level of service

Volume and capacity have same units but capacity Means max. Vol^m at a perticular LOS Whereas Vol^m is the actual Rate of Flow.

Traffic capacity is classified as –

a) Basic capacity :– Max. no. of veh. that can pass a given point in unit time under ideal Roadways and traffic condition is called Basic capacity or theoritical capacity.

NOTE. Two identical Roads will have same Basic capacity

b) possible capacity :– It is the capacity under prevailing condit

c) practicle capacity :– since possible capacity can vary b/w 0 and Basic capacity, for design purpose we adopt an in b/w value such that the traffic density is not so high as to cause unreasonable delays and restrictions. such a capacity is known as practicle capacity or design capacity

** calculation of theortical Max capacity :–

(All calculations are for single lane)

(i) from space headway

$$q = KV$$

$$q = \frac{V}{\text{avg. space headway}}$$

$$C = \frac{V}{(\text{Avg. space Headway})_{Min}}$$

C → veh/hr, V → km/hr, (Avg. space Headway) → km/veh

*
$$C = \frac{1000\,V}{S}$$

S → meter/veh.

C → capacity → veh/hr.

V → avg. Traffic speed in km/hr

S → Min. space headway (m/veh.)

① $S = 0.7\,V_b + L$ ⇒ { V_b → m/sec ; L → 6m if not given }

② $S = SSD + L$ → { $\mu = 0.35 \to 0.4$; $t_r = 2.5$ sec. }

ii) from Time headway

$$q = \frac{1}{(\text{Avg. Time headway})}$$

$$C = \frac{1}{(\text{Avg. Time headway})_{Min.}}$$

C → veh/hr, (Avg. Time headway) → hr/veh

*
$$C = \frac{3600}{H_t}$$

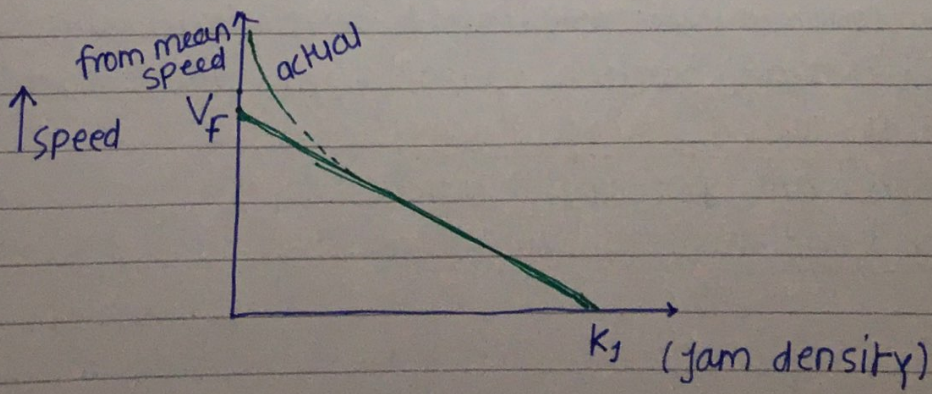
C → capacity → veh/hr

H_t → Min. Time headway in s/veh.

Relation B/W SPEED - DENSITY and FLOW :-

1) Speed - Density Relation

According to Greenshield's assumption speed - density Relation is Linear



$$V = \frac{-V_f}{k_j} k + V_f$$

$$\boxed{V = V_f\left[1 - \frac{k}{k_j}\right]}$$ ***

2) speed - flow relation

$$q = kV$$

$$q = \left[(V_f - V)\frac{k_j}{V_f}\right]V \quad \text{—(i)}$$

* at Max. q we've $\frac{dq}{dv} = 0$

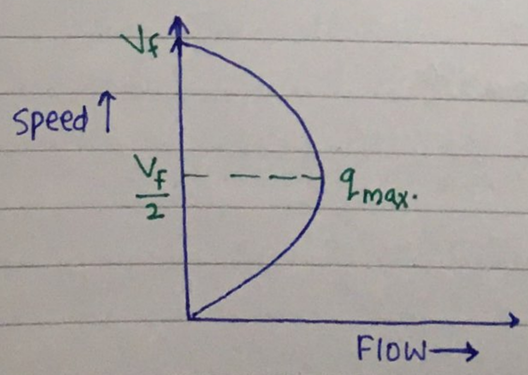
differentiating eqⁿ (i) and substituting $\frac{dq}{dv} = 0$

$$V = \frac{V_f}{2}$$

substituting above value in eqⁿ (i) we've

$$\boxed{q_{max.} = \frac{V_f k_j}{4}}$$

$q_{max.}$ → known as Max. capacity or capacity flow.



3) Flow - density Relation :-

$$q = kv$$

$$q = k\left(\frac{-v_f}{k_j}k + v_f\right) \quad \text{—(ii)}$$

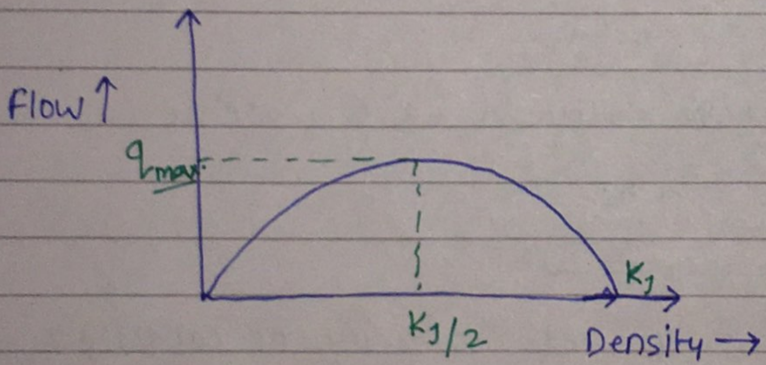
* at Max q we've $\frac{dq}{dk} = 0$

differentiating eqⁿ (ii) and subsbstituting $\frac{dq}{dk} = 0$
we've

$$k = \frac{k_j}{2}$$

substituting above value in eqⁿ (ii)

*
$$\boxed{q_{max} = \frac{V_F k_j}{4}}$$



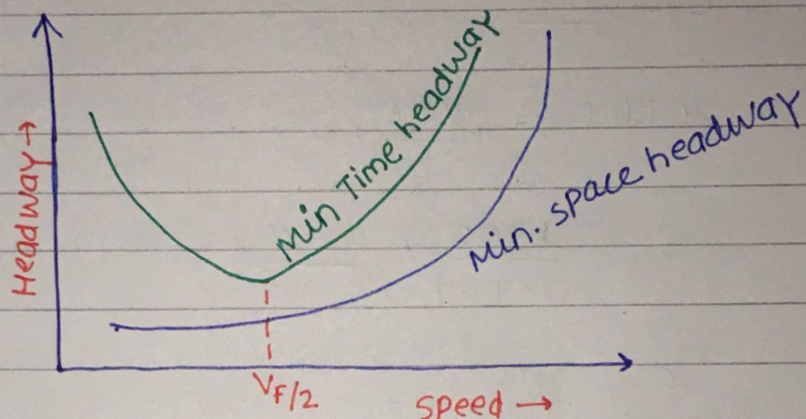
1) $C = \frac{1000V}{S}$ ~~→ q_{max} ≠~~

2) $C = \frac{3600}{H_t}$

3) $q_{max} = \frac{V_f k_j}{4}$

Relation B/W speed and Min. headway :-

i) Speed Vs space Headway and Time headway.



*** Important objective

① Space headway ↑ with speed ↑

② Time headway first ↓ than ↑ with speed ↑

Que. Speed - density relation for a perticular road is given by $V = 42.76 - 0.22K$ Where $V \rightarrow$ kmpH and $K \rightarrow$ veh/km calculate capacity flow, Density @ capacity Flow and speed @ capacity Flow.

$$V = -\frac{V_f}{K_J} K + V_f$$

$$V = -0.22K + 42.76$$

So, $V_F = 42.76$, $\frac{V_F}{K_J} = 0.22 \Rightarrow K_J = 194.36$ veh/km

$$q_{max.} = \frac{V_F K_J}{4} = \frac{42.76 \times 194.36}{4} = 2077.7 \text{ Veh/h}$$

$$\text{speed at } q_{max.} = \frac{V_F}{2} = 21.38 \text{ km/hr}$$

$$\text{Density at } q_{max.} = \frac{K_J}{2} = 97.18 \text{ veh/km}$$

Que: For the previous calculate space headway and time headway corresponding to capacity flow.

$$C = \frac{1000\,V}{S}$$

$$2077.7 = \frac{1000}{S} \times \frac{V_F}{2}$$

$$S = \frac{1000}{2077.7} \times \frac{42.76}{2} = 10.29 \text{ m/veh.}$$

$$C = \frac{3600}{H_t}$$

$$2077.7 = \frac{3600}{H_t}$$

$$H_t = 1.73 \text{ sec/veh.}$$

Que: Free Mean speed = 80 KMPH and under stopped condition avg. spacing b/w vehicles = 6.9 m. Determine the Max. capacity?

$$V_f = 80 \text{ KMPH}$$

$$K_j = \frac{1000}{6.9} = 144.9 \text{ veh/km}$$

$$q_{max.} = \frac{V_F K_j}{4} = \frac{80 \times 144.9}{4} = 2898.55 \text{ veh/hr.}$$

Que: The speed-density relation is found to follow Greenberg's Model. What is the density @ capacity flow and the capacity flow?

$$V = V_F \ln\left(\frac{K_j}{K}\right)$$

$$q = KV = K\left(V_F \ln(K_j/K)\right)$$

$$q = KV_F (\ln K_j - \ln K)$$

$$q = KV_F \ln K_j - KV_F \ln K \quad \text{—(i)}$$

At $q_{max.} \Rightarrow \frac{dq}{dk} = 0$

differentiating eq^n (i) w.r. to K and substituting $\frac{dq}{dk} = 0$

$$0 = V_f \ln K_J - V_f \left(K \cdot \frac{1}{K} + \ln K \times 1 \right)$$

$$0 = V_f \ln K_J - V_f - V_f \ln K$$

$$1 = \ln K_J - \ln K$$

$$1 = \ln (K_J / K)$$

$$\frac{K_J}{K} = e$$

$$\boxed{K = K_J / e}$$

$$q = KV$$

@ $K = K_J/e$ $\quad q = q_{max.}$ and $V = V_f$

$$\boxed{q_{max.} = \frac{V_f K_J}{e}}$$

Que. A Two lane urban Road with one way traffic has a Max. capacity of 1800 Veh/hr under jam condition avg. length occupied by a Veh. is 5 m. speed-density relation is linear. Find the density in Veh/km for a traffic vol^m of 1000 Veh./Hr.

Sol^n. for 2-lane

$q_{max.} = 1800$ Veh/hr

$q = 1000$ Veh/hr

for single lane

$q_{max.} = 900$ Veh/hr

$q = 500$ Veh/hr

$$K_J = \frac{1000}{5} = 200 \text{ Veh/km}$$

$$V_f = \frac{4 q_{max}}{K_J} = 18 \text{ km/h}$$

$$q = KV = K\left(-\frac{V_f}{K_J} K + V_f\right)$$

$$q = \frac{-18}{200} K^2 + 18K$$

$$500 = \frac{-18K^2}{200} + 18K$$

$K = 33.33$ Veh/km & 166.66 Veh/km

for two - lanes

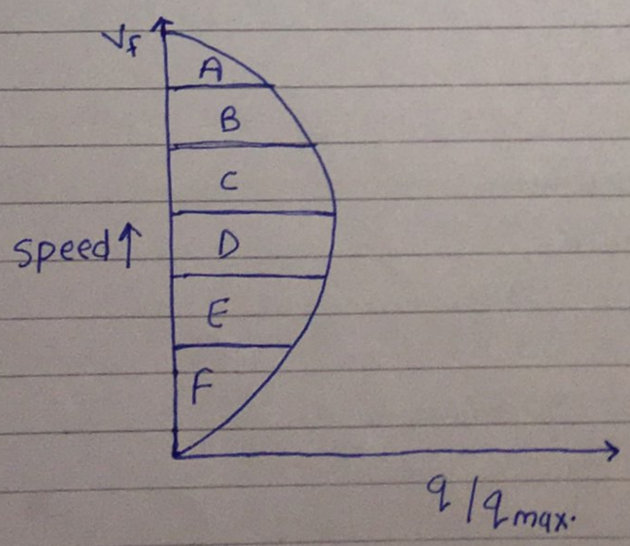
$K = 66.66$ Veh/km & 333.33 veh/km.

LEVEL OF SERVICE (LOS)

capacity is a quantative Measure Whereas Los is a qualitative measure of Flow. LOS tries to explain How good the present traffic conditions are
It is defined on the basic of MOE (Measure of effectiveness) Under Which Generally three parameters are taken -

a) speed and Travel time
b) Density
c) Delay

Highway capacity Manual classifies 6 Los Based on Travel speed and Volume to capacity Ratio



LOS A: Driver has complete freedome and is at highest physcological comfert. Avg. speed is about 90% of Free mean speed and avg. spacing b/w Vehicles is about 167 m

LOS B There is still regionably free flow condition. Avg. speed is about 70% of free Mean speed and Avg. spacing b/w vehicles about 100 m

LOS C Avg. speed → 50%
Avg. spacing → 67 m

LOS D speed begins to decline.
Avg. speed → 40%
Avg. spacing → 50 m

LOS E There is almost no usable Gap

LOS F It is also known as Brake down zone. in this if one veh. Brakes down it falls a large queue.

⑥ PARKing study :-

1) off street parking
2) on street parking (kerb parking)

Types of on street parking —

a) parallel parking :-

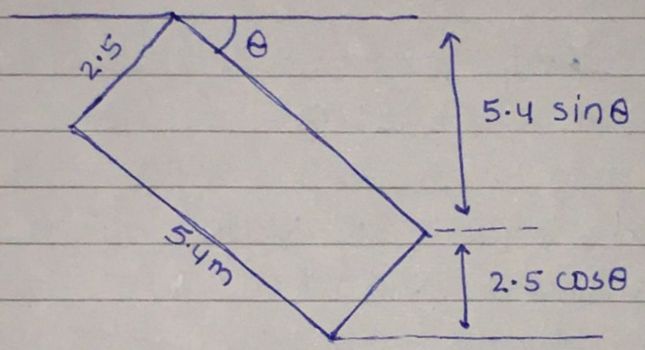
- used when there is width restriction
- least accident with main traffic
- parking Maneuver is difficult
- least no. of veh. per unit length of Road.

b) Angled parking

Generally 30°, 45°, 60°, 90° Angle parking is used

90° parking → • Max. No. of veh. per unit length of Road
• Width Requirement is more
• Max. chances of accident with Main traffic

NOTE 45° Angle parking is used for Most optimum Result.



Width req. = $5.4\sin\theta + 2.5\cos\theta$

⑦ ACCIDENT ANALYSIS :-

Accident studies are used to find out the reason behind accidents and to take preventive measures in terms of design and control

Various records that are Maintained in these studies are —

1) Location file :→ points of high accidents are Noted
2) Spot Map :→ Accident locations are shown in a Map
3) condition Diagram :→ Drawings are prepared to scale showing all the imp. Physical conditions at the time of accident
4) Collission Diagram :- Diagram showing path of vehicle and pedestrain involved in accident

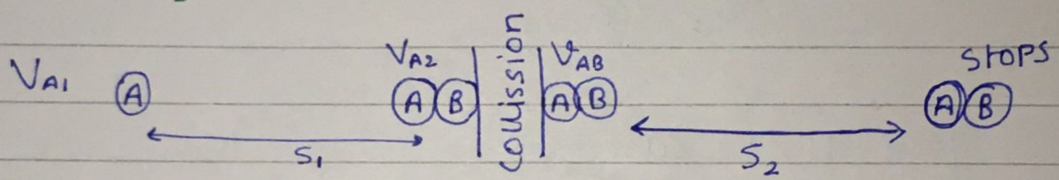
Scanned with CamScanner

For analysis purpose we calculate the initial velocities of vehicles involves in accident. Following assumptions are made –

a) When skid marks are present → 100% skidding is assumed and if skid marks are not assumed → free collussion is assummed.

b) When vehicles are on same path plastic collission is assumed

Case-I Collission of a moving veh. with parked veh. along the line.



$$V^2 = u^2 + 2as$$
$$0 = V_{AB}^2 - 2\mu g S_2$$

$$V_{AB} = \sqrt{2\mu g S_2}$$

Applying Momentum conservation just before and after collission

$$m_A V_{A2} = (m_A + m_B) V_{AB}$$

$$\boxed{V_{A2} = \frac{m_A + m_B}{m_A} V_{AB}}$$

$$V^2 = u^2 + 2as$$
$$V_{A2}^2 = V_{A1}^2 - 2\mu g S_1$$

$$\boxed{V_{A1} = \sqrt{V_{A2}^2 + 2\mu g S_1}}$$

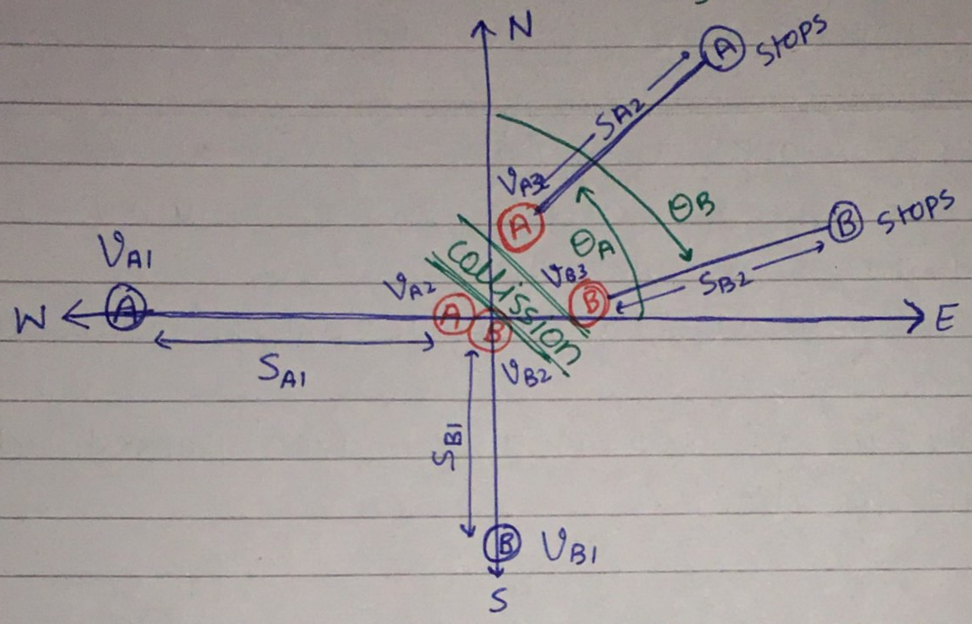
V_{A1} → initial speed of A. V_{A2} → speed of A just before collission

V_{AB} → Combined speed of A & B just after collission

S_1, S_2 → skid marks before and after collission

m → mass

Case-II Collission of Veh. moving at Right Angles.



$$V^2 = u^2 + 2as$$

For A:

$$0 = V_{A3}^2 - 2\mu g S_{A2}$$

$$V_{A3} = \sqrt{2\mu g S_{A2}}$$

For B:

$$V_{B3} = \sqrt{2\mu g S_{B2}}$$

From momentum conservation —

X-axis

$$m_A V_{A2} = m_A V_{A3} \cos\theta_A + m_B V_{B3} \sin\theta_B$$

$$V_{A2} = \frac{m_A V_{A3} \cos\theta_A + m_B V_{B3} \sin\theta_B}{m_A}$$

Y-axis

$$V_{B2} = \frac{m_B V_{B3} \cos\theta_B + m_A V_{A3} \sin\theta_A}{m_B}$$

$$V^2 = u^2 + 2as$$

$$V_{A2}^2 = V_{A1}^2 - 2\mu g S_{A1}$$

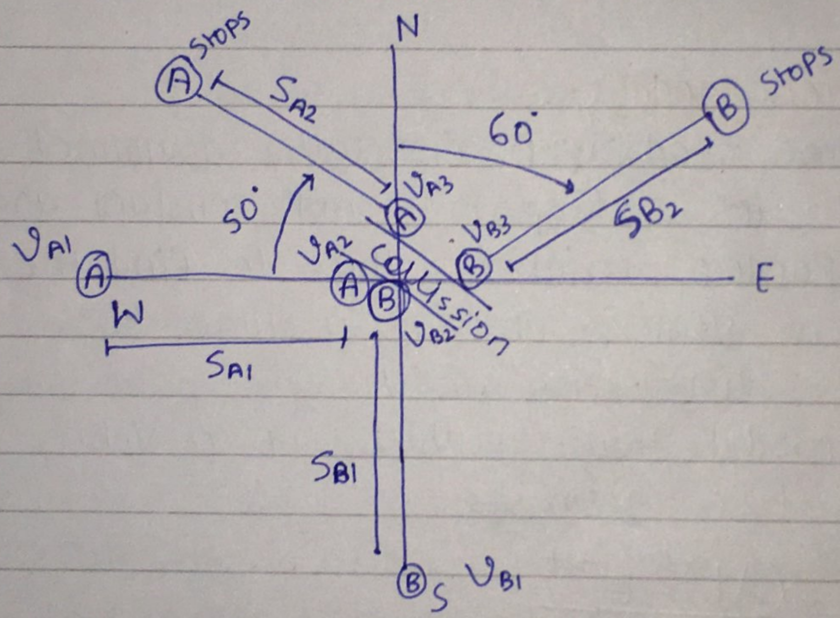
$$V_{A1} = \sqrt{V_{A2}^2 + 2\mu g S_{A1}}$$

$$V_{B1} = \sqrt{V_{B2}^2 + 2\mu g S_{B1}}$$

Scanned with CamScanner

Que. Two vehicles approaching at Right angles, A from West and B from South collide and after collission A skids 50° N of West and B skids 60° E of N. The Data recorded in Accident analysis is given below calculate initials speeds if $\mu = 0.55$

	A	B
S_1	35 m	20 m
S_2	15 m	30 m
m	4.5 tonne	6 tonne



$$V^2 = u^2 + 2as$$

$$0 = V_{A3}^2 - 2\mu g S_{A2}$$

A → $V_{A3} = \sqrt{2\mu g S_{A2}} = \sqrt{2 \times 0.55 \times 9.81 \times 15} = 12.72$ m/sec

B → $V_{B3} = \sqrt{2\mu g S_{B2}} = \sqrt{2 \times 0.55 \times 9.81 \times 30} = 17.99$ m/sec.

From momentum conservation

X-Axis $\quad m_A V_{A2} = -m_A V_{A3} \cos 50° + m_B V_{B3} \sin 60°$

$V_{A2} = 12.6$ m/sec

Y-Axis $\quad m_B V_{B2} = m_B V_{B3} \cos 60° + m_A V_{A3} \sin 50°$

$V_{B2} = 16.31$ m/sec

$$v^2 = u^2 + 2as$$

$A \rightarrow \quad v_{A2}^2 = v_{A1}^2 - 2\mu g S_{A1}$

$$v_{A1} = \sqrt{v_{A2}^2 + 2\mu g S_{A1}}$$

$$= \sqrt{12.6^2 + 2 \times 0.55 \times 0.98 \times 35}$$

$$v_{A1} = 23.16 \text{ m/sec}$$

$$v_{B1} = \sqrt{v_{B2}^2 + 2\mu g S_{B1}} = 21.95 \text{ m/s.}$$

Poisson's Distribution Model :-

In calculations time headway is generally assummed constant However in reality it is not constant and it follows a Random variation. Thus to find the probability of n vehicles arriving in time t we use poisson's distribution model.

As per this model the probability of n veh. arriving in time t is given as

$$P(n) = \frac{(\lambda t)^n e^{-\lambda t}}{n!}$$

Where $P(n) \rightarrow$ prob. of n veh. arriving in time t sec.

$\lambda \rightarrow$ vehicular flow in veh/sec.

Que. An observer counts 360 veh. per hour at a specific highway location. Assuming that arrival of veh. @ this location follows poisson's distribution. Estimate the prob. of having 0, 1, 2, 3, 4, 5 and more veh. arriving over 20 sec. time interval.

$$\lambda = \frac{360}{3600} = 0.1 \text{ veh/sec}$$

8/10/2017

$$t = 20 \text{ sec.}$$

$$P(0) = \frac{(0.1 \times 20)^0 e^{-0.1 \times 20}}{0!} = 0.135$$

$$P(1) = \frac{(0.1 \times 20)^1 e^{-0.1 \times 20}}{1!} = 0.27$$

$$P(2) = \frac{(0.1 \times 20)^2 e^{-0.1 \times 20}}{2!} = 0.27$$

$P(3) = 0.18$

$P(4) = 0.09$

$P(0) + P(1) + P(2) + P(3) + P(4) + P(\geq 5) = 1$

$P(n \geq 5) = 1 - P(0) - P(1) - P(2) - P(3) - P(4)$

$P(n \geq 5) = 0.055$

probability of 0 veh. arriving in t sec is same as the probability of Time headway being $\geq$ t sec.

$$P_{(h_t \geq t)} = P(0) = e^{-\lambda t}$$

This distribution of time headway is known as -ve exponential distribution or simply exponential distri.

Que. for previous que. calculate the prob. that

a) Time headway < 8 sec

b) Time headway $\geq$ 10 sec.

c) Time headway b/w 8-10 sec.

a) $P_{(h_t < 8)} + P_{(h_t \geq 8)} = 1$

$$P_{(h_t \geq 8)} = e^{-\lambda \times 8} = e^{-0.1 \times 8}$$

$$P_{(h_t < 8)} = 1 - 0.45 = 0.55$$

b) $P_{(h_t \geq 10)} = e^{-\lambda t} = e^{-0.1 \times 10} = 0.368$

$$P_{(h_t < 8)} + P_{\{h_t \in [8,10)\}} + P_{(h_t \geq 10)} = 1$$

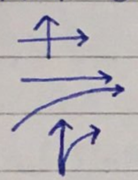
$$P_{\{h_t \in [8,10)\}} = 1 - 0.55 - 0.368$$
$$= 0.081$$

TRAFFIC CONTROL AND REGULATIONS

Traffic Intersection:- Area where 2 or more roads joins or crosses

At traffic intersection change in dir^n of movement may occure. Due to movement of traffic at intersection various types of conflict occur which are:-

1) Crossing conflict
2) Merging conflict
3) Diverging conflict



① **Crossing conflict:-** Considered as major conflicts various types of crossing conflicts are:-

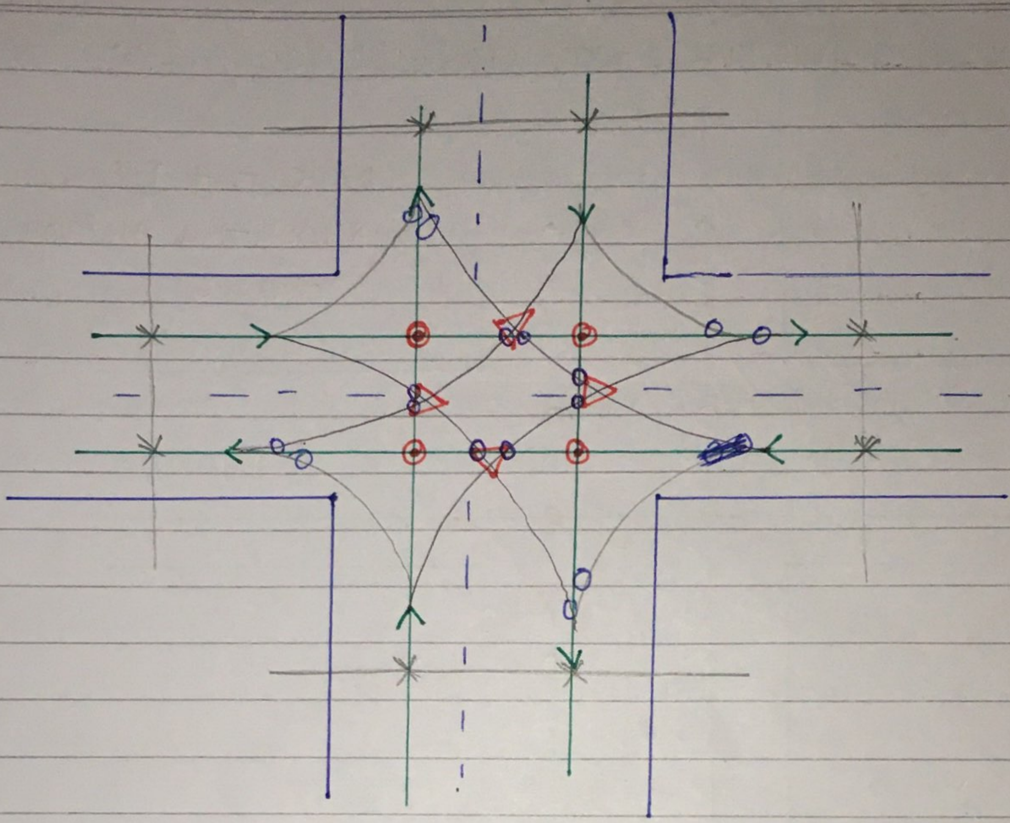
a) Through - Through crossing
b) Right Through crossing
c) Right - Right crossing

② **Merging / Diverging conflict:-** Considered as Minor conflicts because of lower relative velocities due to small intersection angles

Diverging conflicts are all together neglected because of their lower velocities along with small intersection angles.

Conflicts at various Intersections:-

Case-I When a 2-lane 2-way Road crosses 2-lane 2-way Road



<u>Major</u>

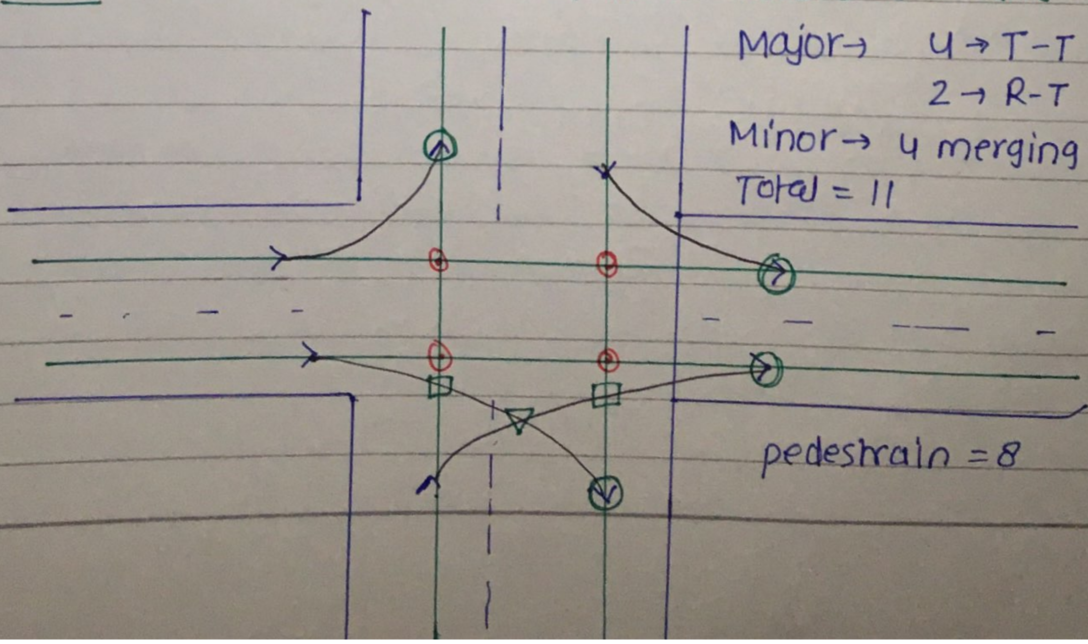
4 through - through crossing conflict
8 Right - through crossing conflict
4 Right - Right crossing conflict

<u>Minor</u> 8 - merging conflict

Total = 16+8 = 24

pedestrain conflict = 8

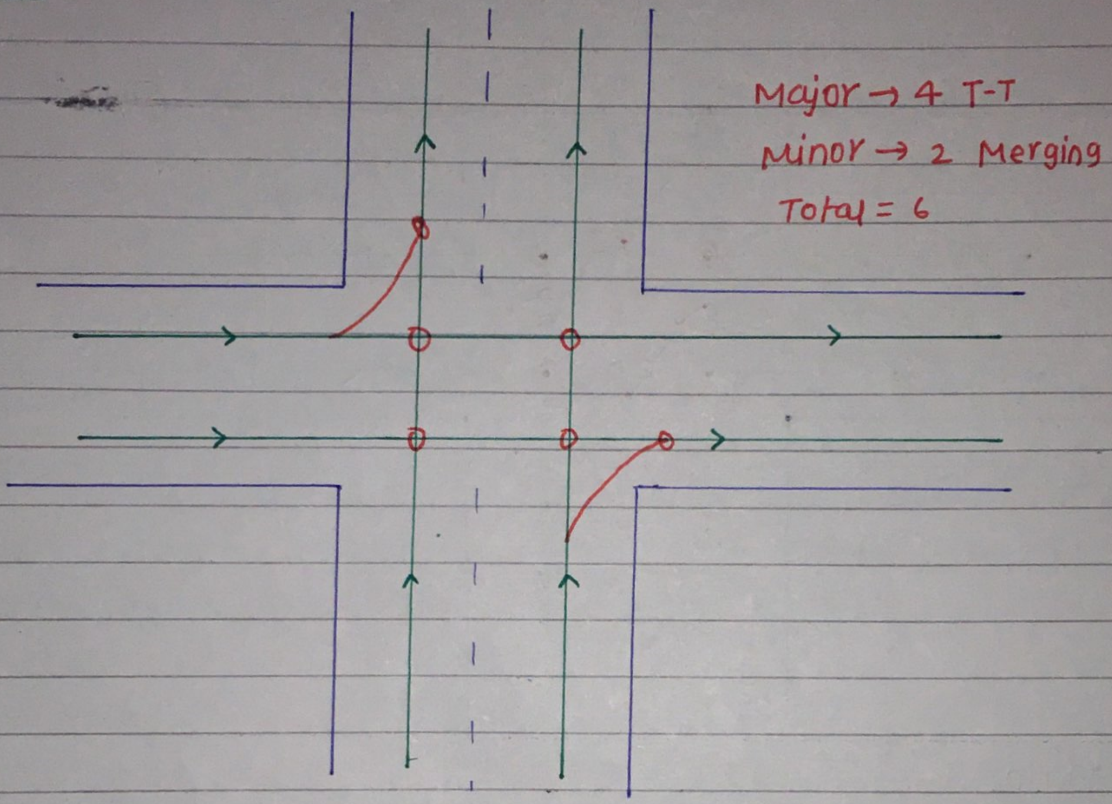
<u>Case-II</u> When 2-lane 2-way Road crosses a 2-lane-1 way Road.



Major → 4 → T-T
2 → R-T , 1 → R-R
Minor → 4 merging
Total = 11

pedestrain = 8

Case-III When a two lane 1 way crosses 2-lane, 1-way Road

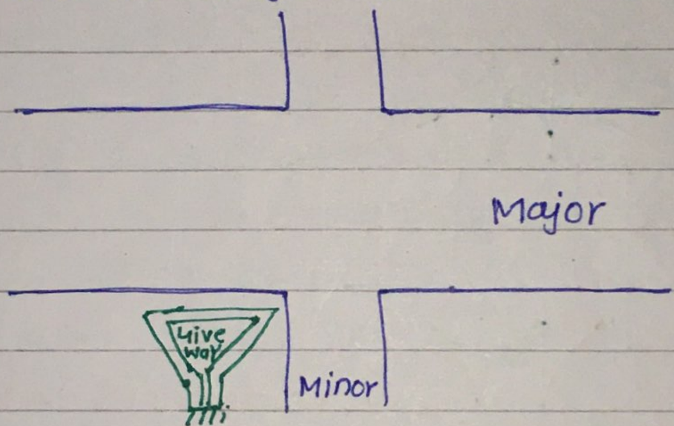


Intersection control

purpose is to reduce conflicts on intersections, it is categroied as -

1) passive control :- When traffic Vol^m is less no exclusive control is required. Road users are req. to Follow traffic Rules. Traffic signs and Road marking are use to compliment the control.

For ex: Give way control is established with the req. Minor road at proirity intersections to slow down and allow the Major Road to proceed.



2) semi-control :- or partial control :- Driver are gently Guided to avoid conflicts. Channelization and rotary comes under this category.

3) Active control :- Road users are forced to follow the path suggested by traffic control agency. Traffic signals and Grade seprated intersections comes under this category.

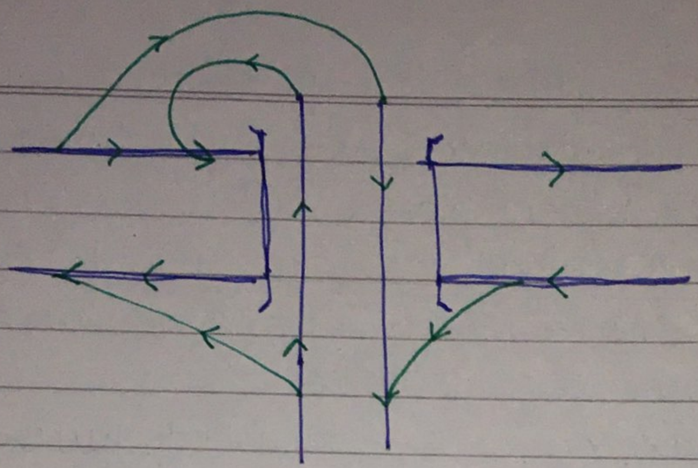
Types of Intersections :-

① Grade seprated intersections :- Grade seprated intersections can be overpass, underpass or interchange

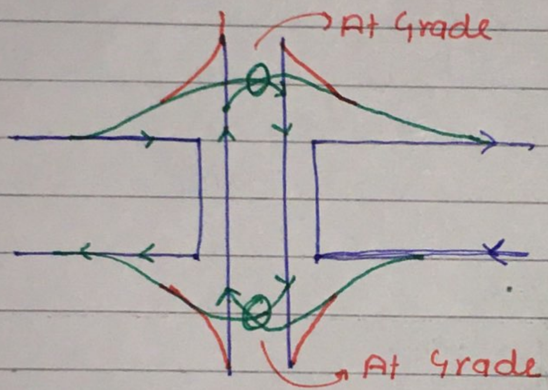
Interchange is a Grade seprated intersection with connecting roadways and ramps for turning traffic ~~an~~ b/w highway approaches

various types of interchanges are :-

i) trumpet interchange :- used for 3 legged intersection (T intersection or Y-intersection)

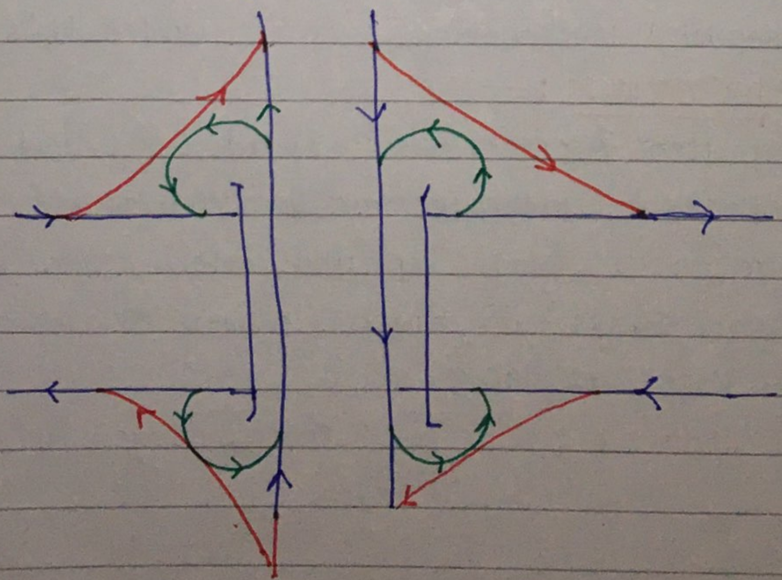


ii) Diomond interchange:- it is the simplest kind of 4 legged intersection. it is used when a Major Road crosses a Minor Road.



Because of At Grade intersection all crossing conflicts are not eliminated and all mergings are not left merging.

iii) Clover leef :- (Full) It is used when a Major Road crosses a Major Road.

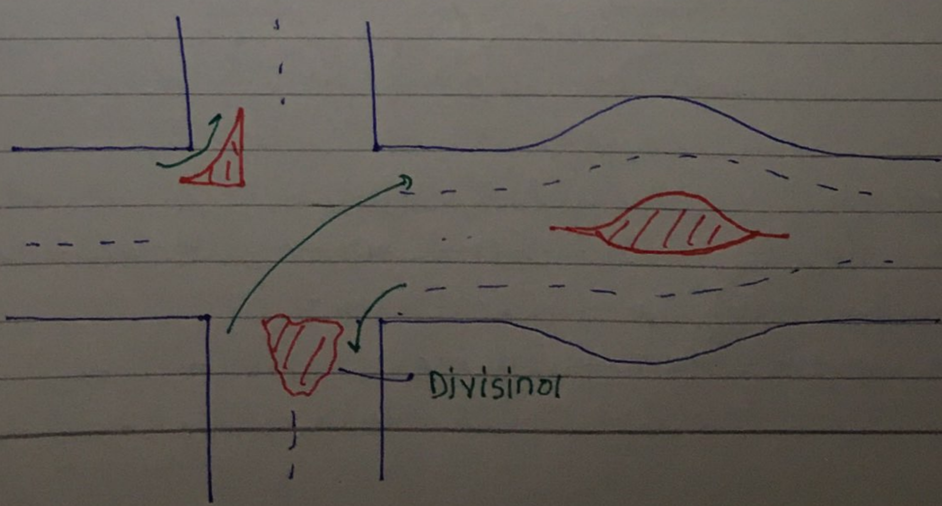


All crossing conflicts are eliminated and all mergings are left mergings
Area req. is more and is expensive

NOTE: partial clover reef is used when a Major Road crosses a Minor Road.

iv) Rotary interchange :- used when multiple Roads intersects
v) Directional interchange

② At Grade intersection:-
All Roads of intersection meet at about same level.
It is classified as A) unchannelised intersection.
B) channelised intersection → channelization is done by providing traffic Islands.
various Traffic Ilands are
i) Divisional Island
It seprates opposite flow of traffic and prevents head on collission
ii) channelising Island
Traffic is Gently Guided into proper channel
iii) Pedestrian Refuse Island
iv) pedestrian loading Island
It is provided near Bus stop for loading unloading of public transport



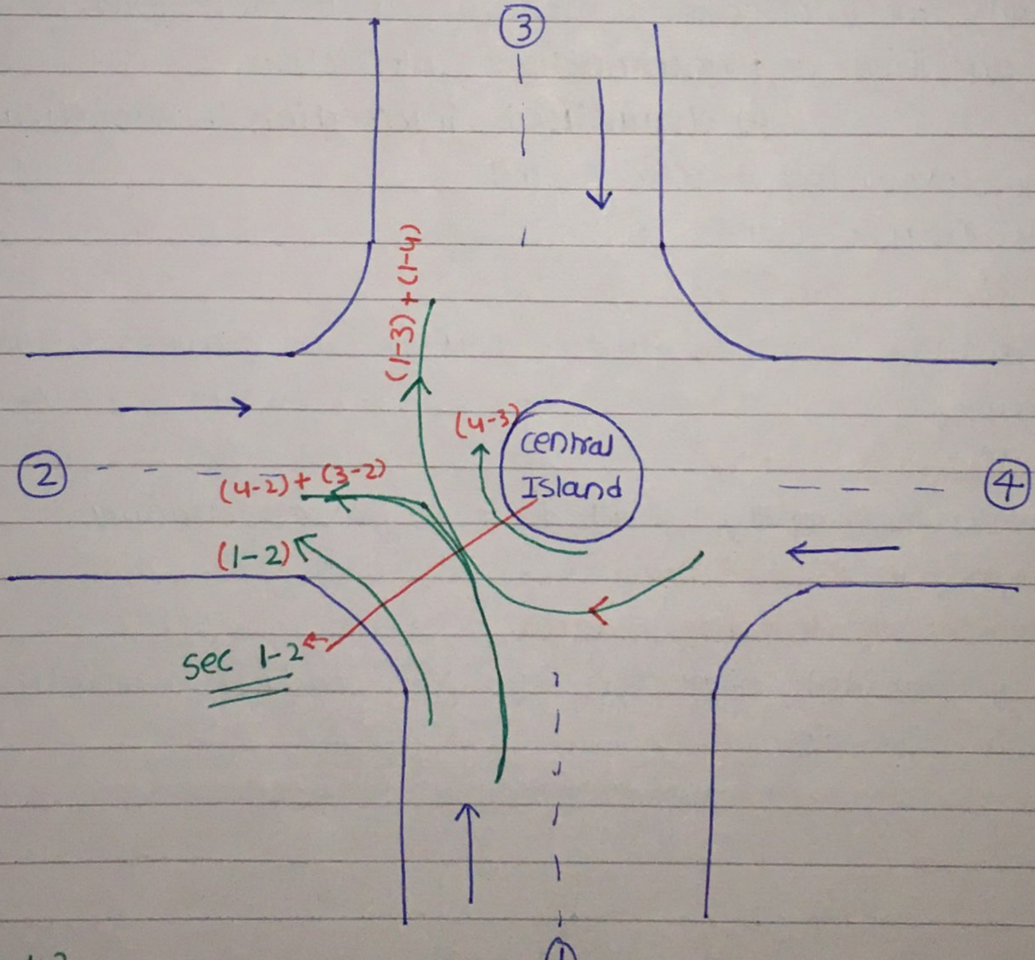
c) Rotary intersection

Rotary is a type of at grade intersection which converts all the Major conflicts into Minor conflicts (crossing conflicts are converted into weaving conflict) vehicles entering the rotary are gently forced to move in clockwise dir^{n} in an orderly way and then weav out in desired dir^{n}. Traffic operation at rotary are -

I) Merging

II) diverging

III) Weaving (combination of Merging & diverging)



sec. 1-2

Total traffic = (1-2) + (1-3) + (1-4) + (3-2) + (4-2) + (4-3)

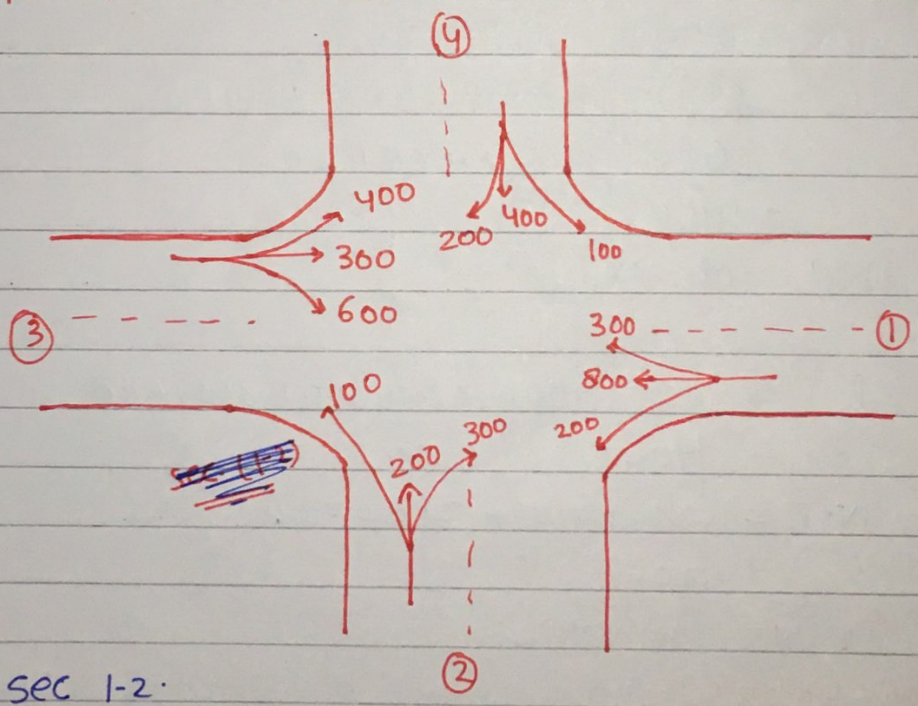
Weaving = (1-3) + (1-4) + (3-2) + (4-2)

Non. weaving = T.T − weaving = (1-2) + (4-3)

* Weaving Traffic @ 1-2 = 1 to all dir^n except 1→2
+ all dir^n to 2 except 1→2

Central Island is the large central part of the rotary. It converts Major conflicts into Minor conflicts. It is achived by reducing the intersection angle of conflicting Vehicles thereby reducing their relative velocities and thus the sevearity of accidents.

Que. Find Weaving and Non-Weaving traffic at all sec^n for the intersection shown below:-



sec 1-2.

T.T = (1-2) + (1-3) + (1-4) + (3-2) + (4-2) + (4-3)
= 200 + 800 + 300 + 600 + 400 + 200
= 2500

W.T = (1-3) + (1-4) + (3-2) + (4-2)
= 800 + 300 + 600 + 400
= 2100

N.W = 2500 - 2100 = 400 veh/hr.

Sec^n 2-3

T·T = (2-1) + (2-3) + (2-4) + (4-3) + (1-2) + (1-3)
= 100 + 200 + 300 + 200 + 800 + 300
= 1900

W·T = (2-1) + (2-4) + (1-3) + (4-3)
= 300 + 200 + 800 + 200
= 1500

N·W = 400 veh/hr

Sec^n 3-4

T·T = (3-1) + (3-2) + (3-4) + (1-4) + (2-1) + (2-4)
= 400 + 300 + 600 + 300 + 300 + 200
= 2100

W·T = (3-1) + (3-2) + (1-4) + (2-4)
= 600 + 300 + 300 + 200
= 1400

N·W = 700 veh/hr

Sec^n 4-1

T·T = 200 + 400 + 100 + 300 + 300 + 600
= 1900

W·T = 200 + 400 + 300 + 300
= 1200

N·W = 700 veh/hr

Advantages of Rotary:-

1. Major conflicts are converted in minor
2. self Governing
3. vehicles doesn't need to stop

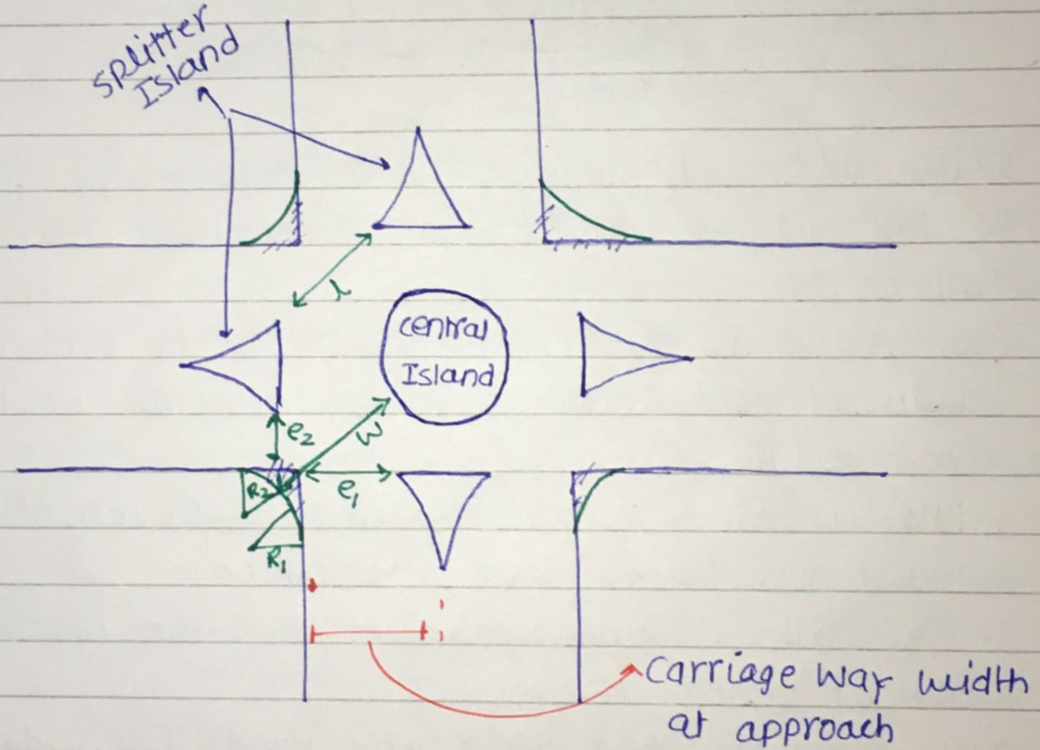
Disadvantages:-

1. requires large Area
2. Non-suitable for pedestrain crossing as veh. doesn't stop
3. speed of veh. is reduced even when traffic vol. is low.

Guidelines of Traffic Rotary :-

1) suitable when traffic from all approaches are equal
2) upper limit of total volume is about 3000 veh/hr , lower volume is 500 veh/hr
3) When proportion of Right turning traffic is large (>30 %) Rotary becomes more benificial than signalized intersection

DESIGN ELIMENT OF ROTARY :-



e_1 → entry width
e_2 → exit width
W → width of weaving section
l → weaving length
R_1 → entry Radius
R_2 → exit Radius

$(R_1 < R_2)$

i) Design speed :- It is taken as 30 kmph → Urban Rotary
40 kmph → Rural Rotary

ii) Entry Radius, Exist Radius and Radius of central Island :-
Entry to Rotary is not straight but a small curvature is introduced which will force the drivers to reduce their speed.
Entry Radius for urban rotary - 20 m
for Rural rotary - 25 m (approx)

$$\frac{v^2}{gR} = \mu \leftarrow (0.43 - 0.47)$$

Exit radius should be greater than entry Radius so that vehicles can discharge from the rotary at a higher rate.
Exis radius is generally taken 1.5 to 2 times Entry Radius.
Radius of central Island is approximately 1.33 times entry Radius

iii) Width at Entry & Exit :- Width is governed by traffic entering and leaving the intersection
- Entry width should be lower than the carriage way width at approach.

IRC suggests that a 2 lane road 7 m wide (approach carriage way) should be kept at 7m for urban rotary and 6.5 m for Rural Rotary.

*** NOTE: if approach carriage way is of 14 m entry width can be adopted as 10 m (Urban) and 8 m (Rural)

iv) Weaving width and Weaving length :- Width of weaving section should be higher than the width at entry than exit. For design it is taken as 1 lane more than the avg. of entry & exis width

$$W = \frac{e_1 + e_2}{2} + 3.5$$

Weaving length determines How smoothly veh. can merge and diverge, Normally weaving length is taken as 4 times of weaving width.

$$l = 4W$$

$$0.12 \le \frac{W}{l} \le 0.4$$

larger is the weaving length greater is the tendency of speeding.

NOTE. For smooth flow of traffic weaving angle (Angle b/w veh. entering and leaving rotary on adjacent Roads) should be as small as possible but not smaller than 15° o/w the central Island will have to be very large.

5) Capacity of Rotary:- overall capacity of rotary is reported as min. capacity of all weaving section.

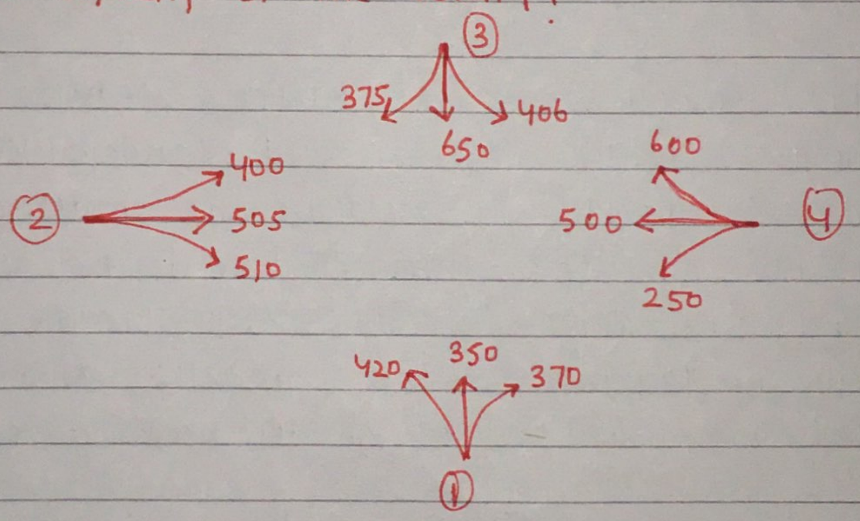
$$Q = \frac{280W\left(1 + \frac{e}{W}\right)\left(1 - \frac{p}{3}\right)}{\left(1 + \frac{W}{l}\right)}$$

Q → capacity in weaving secⁿ in pcu/hr.
W → weaving width
e → avg. of entry and exit width
l → Weaving length
p → proportion of weaving traffic or weaving Ratio

$$p = \frac{\text{Weaving traffic}}{\text{Total traffic}}$$

NOTE: if e, W, and l for all weaving secⁿ are same then the largest value of p is calculated of all weaving secⁿ and corresponding capacity (Min. capacity) is reported as overall capacity of Rotary.

Que. Turning movement studies at a intersection gave following results :-
Width of carriage way at approach to intersection = 14m
entry and exist width are 10m each.
Find the capacity of the rotary?



$$e = \frac{e_1+e_2}{2} = \frac{10+10}{2} = 10\ m$$

$$W = \frac{e_1+e_2}{2} + 3.5 = 13.5\ m$$

$$l = 4W$$

$$l = 54\ m$$

sec 1-2

T.T = 420 + 350 + 370 + 375 + 500 + 600 = 2615 Veh/hr

W.T = 350 + 370 + 375 + 500 = 1595 Veh/hr

p = weaving ratio = 0.61

sec 2-3 T·T = 400 + 505 + 510 + 600 + 350 + 370 = 2735

W·T = 505 + 510 + 350 + 600 = 1965

p = Weaving Ratio = ~~0.719~~ 0.718

sec- 3-4 T·T = 375 + 650 + 406 + 370 + 505 + 510 = 2816

W·T = 375 + 650 + 370 + 505 = 1900

p = Weaving Ratio = 0.675

sec- 4-1

T·T = 600 + 500 + 250 + 510 + 650 + 375 = 2885

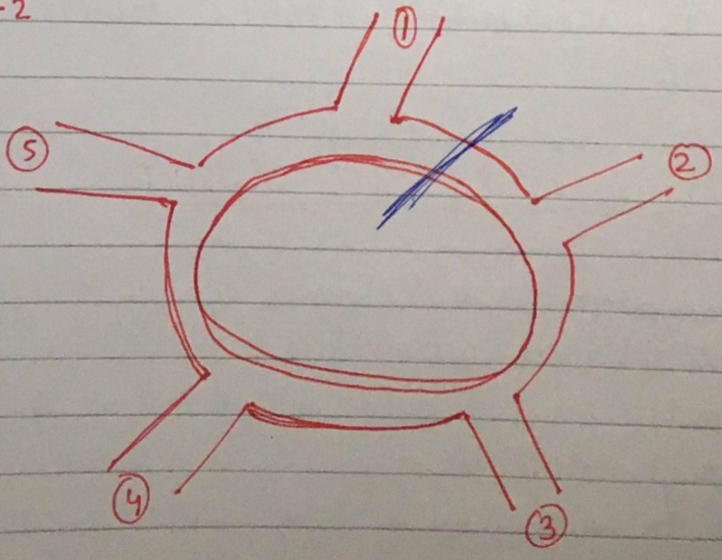
W·T = 500 + 600 + 510 + 650 = 2260

p = 0.783

$$Q = \frac{280 \times 13.5 \left(1 + \frac{10}{13.5}\right)\left(1 - \frac{0.783}{3}\right)}{\left(1 + \frac{13.5}{54}\right)}$$

Q = 3890.09 veh/hr

Que. A Road intersection has 5 legs designated as 1, 2, 3, 4, 5 as shown below. Traffic vol^m in pcu/hr is given below. calculate the Weaving Ratio for sec^n 1-2



$V_{1-2} = 37$ | $V_{3-1} = 466$ | $V_{4-1} = 180$ | $V_{5-1} = 45$

$V_{1-3} = 303$ | $V_{3-2} = 122$ | $V_{4-2} = 54$ | $V_{5-2} = 132$

$V_{1-4} = 64$ | $V_{3-4} = 47$ | $V_{4-3} = 18$ | $V_{5-3} = 62$

$V_{1-5} = 52$ | V_{3-5} 657 | $V_{4-5} = 116$ | $V_{5-4} = 15$

Sec^n 1-2

T.T = 37 + 303 + 64 + 52 + 122 + 18 + 54 + 132 + 62 + 15

T.T = 859 Veh/hr

W.T = 303 + 64 + 52 + 122 + 54 + 132

W.T = 727 veh/hr

$$p = \frac{W.T}{T.T} = \frac{727}{859} = 0.859$$

TRAFFIC SIGNALS :-

Traffic signals are classified as

1) Fixed Time signal or pre time signal :-
signal timing is independent of traffic vol^m (in real time)

2) semi-actuated signal :-
signal timing is influenced by traffic in some dir^n and is not fully dependent on traffic Volume.

3) Fully Actuated signal :-
signal time is controlled by traffic volume on all approaches.

Type of co-ordinations of Traffic signal :-

1) simultaneous systems :- All signals along the given Road show same indication at same time

2) Alternate system :- Alternate signals show opposite indication along the route at same time
It is found to be more satisfactory than simultaneous system.

iii) simple progressive system :-

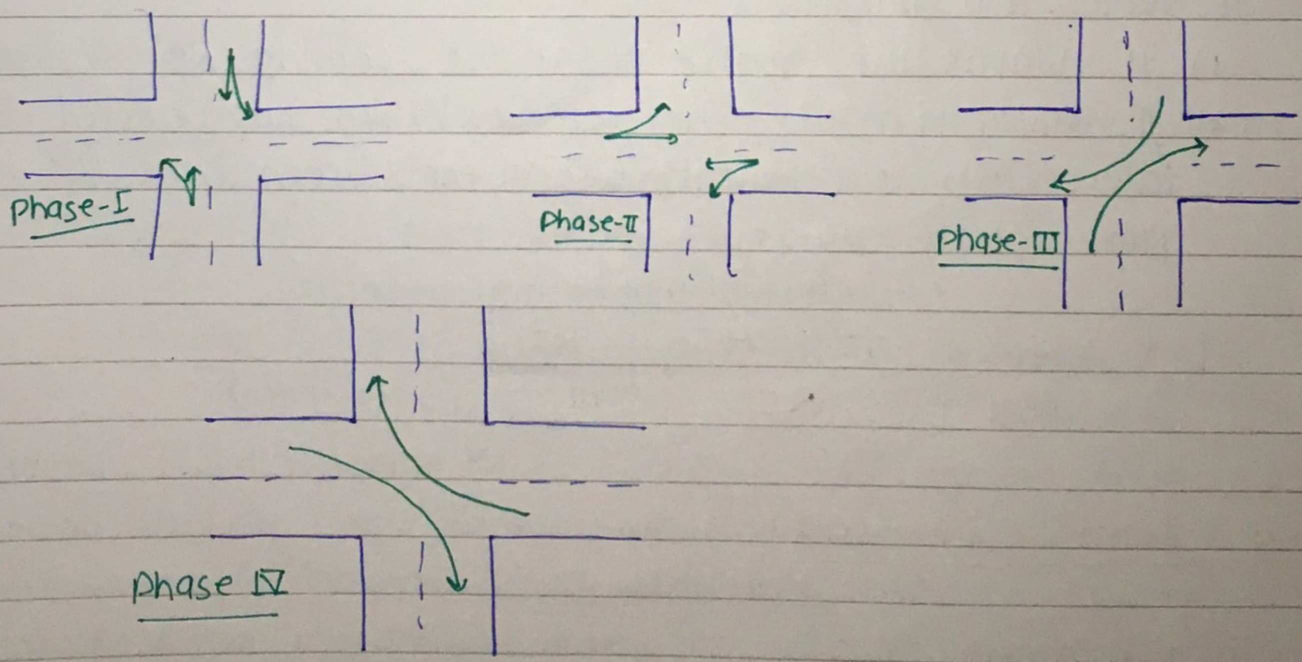
A time shedule is made to permit as nearly as possible a contineous operation of Group of vehicles along the Main Road at a reasonable speed.

The phases and intervals at each signals may be different but each signal unit works as fixed time signal

iv) Flexible progressive system :-

In this system it is possible to automatically vary cycle length, cycle devision and the time shedule at each intersection with the help of a computer

Traffic phase :- no. of steps in which traffic is cleared at an intersection. To clear a 4-legged intersection we require a 4-phase signal system



NOTE

Two phase signal @ 4-legged intersection is designed With the Mindset that right-turning traffic is very low

SIGNAL CYCLE :- It is a one complete rotation to all indication provided (Green, Amber and RED)

cycle length :- It is the time in sec.. it takes to complete one full Rotation of indications that is time b/w starting of Green time for one approach till the next time Green signal starts on that approach.

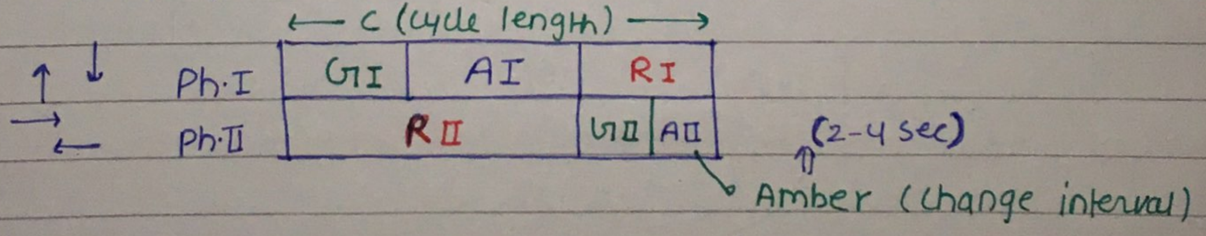
Intervals :- Intervals represents change from one stage to other. There are two types of intervals.

a) change interval
b) clearence interval

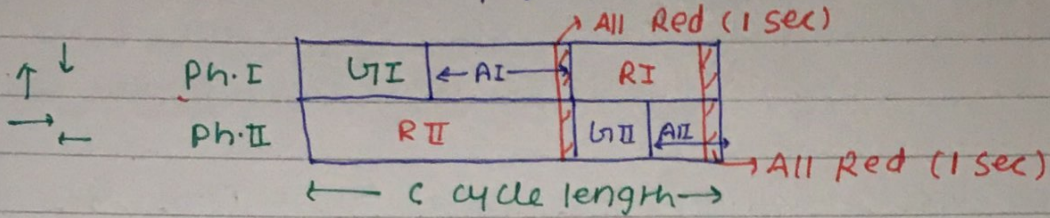
a) change interval :- It is also known as yellow time clearance amber or simply amber

It serves two purposes-

1) It warns the traffic about the coming Red signal
2) It gives time for the traffic which has entered the intersection to clear it before the Green time for the Next phase starts.

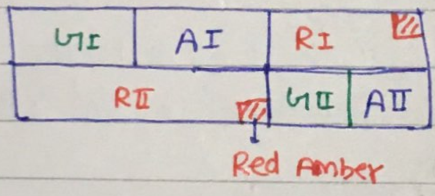


b) clearance interval :- clearance interval is also known as all Red interval. it is included after each Yellow interval and it indicates a period during which all signal phases show Red signal. It is used for clearing the vehicle from intersection. All Red interval is optional and generally provided at large intersections only.



NOTE. For calculations all Red provided for vehicles is taken as a part of Amber

Initial Amber or RED AMBER :- sometimes towards the end of Red signal Amber signal May be put on along with the Red signal in order to indicate 'Get set go' condition. It is actually the last portion of Red signal itself.

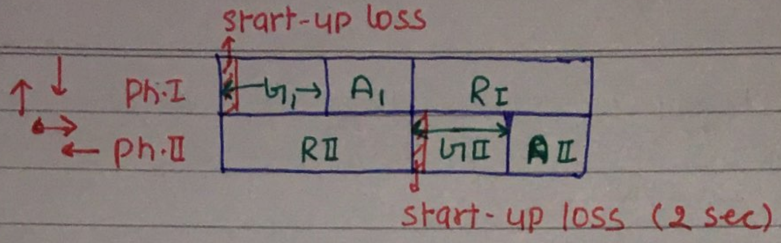


GREEN TIME :- The actual duration for which green signal is turned on called as Green time

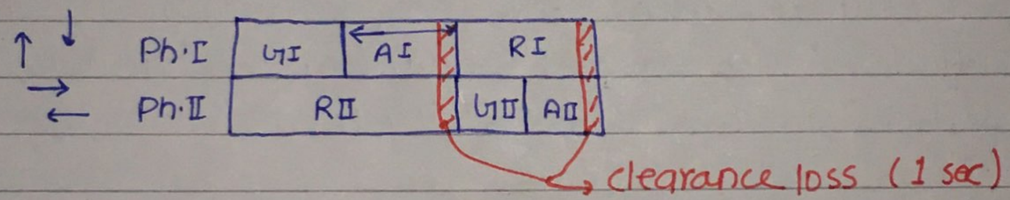
RED TIME :- The actual duration for which Red signal is turned on called as Red time

LOSS TIME :- Due to traffic signal traffic stream is contineously being started and stopped. everytime this happens some portion of the cycle length is not completly utilised and this results in time loss. Total loss can be thought of as

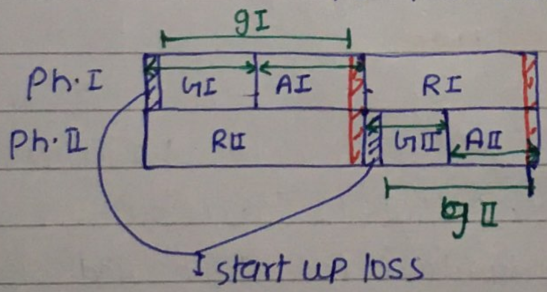
a) start-up loss : When indication changes from Red to Green driver's Don't start instantly. it is due to the reaction time of driver and is accounted for in start-up loss



b) Clearance loss :- When the signal turns from Green to Yellow the later portion of Amber during this interval is generally not utilized by traffic. Also if all red time is given then there is no traffic movement in this time. These two losses are together called as clearance loss.



EFFECTIVE GREEN TIME :- It is the actual time available for the vehicle to cross the intersection.



$$g_I = G_I + A_I - t_{lI}$$

g_I → effective Green for phase I

G_I → Green time for phase I

A_I → Amber time for phase I

T_{lI} → losses in phase one (i.e startup & clearance loss)

$$C - (t_{lI} + t_{lII}) \Rightarrow \text{eff. Green time.}$$

Capacity of a lane :-

① let the cycle time be C sec.

② No. of cycles in one hour = $\frac{3600}{c}$

Hence effective Green time in one hour for a perticular phase = $\boxed{\frac{3600}{c} \times g}$

Hence no. of veh. crossing in 1 hr. per lane that is capacity = $\boxed{\frac{3600 \times g}{h \times c}}$

Where s → saturation flow per lane $\left(\frac{3600}{h}\right)$

Where h → Saturation time headway (that is time headway a contineous movement of traffic)

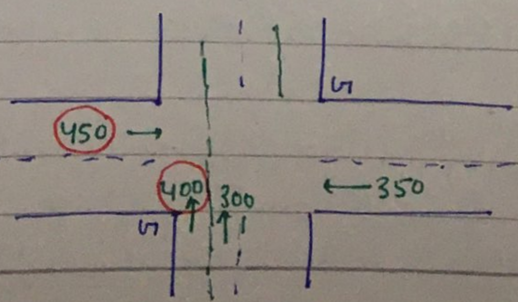
$\frac{g}{c}$ → called 'Green Ratio'

where g → eff. Green time in a perticular phase in 1 cycle

c → cycle time.

Critical lane volume :-

During any Green signal several lanes are permitted to move. one of these lanes will have the most intense traffic. Thus if sufficient time is allocated for this lane then the vehicles on other lane will also be cleared. volume of lane with most intense traffic is called critical lane volume.



Determination of cycle length :-

let the cycle length be c sec.. no. of cycles in 1 hr $= \frac{3600}{c}$, Total loss time in one hour $= \frac{3600}{c}\sum_{1}^{n} t_{li}$

Where $n \rightarrow$ no. of phases.

$t_{li} \rightarrow$ loss in i^{th} phase / cycle

If losses in all phases are same ~~then~~ (say t_l)

then total loss in one hour $= \frac{3600}{c}(n t_l)$

eff. Green time in one hour $= 3600 - \frac{3600}{c}(n t_l)$

Thus in effective Green time of one hour we must clear critical lane volumes of all phases

$$3600 - \frac{3600}{c}(n t_l) = V_c \times h \quad -①$$

{ V_c = sum of critical lane vol. of all phases

$$V_c = \sum_{i=1}^{n} V_{ci}$$

$V_{ci} \rightarrow$ critical lane volume for i^{th} phase

$h \rightarrow$ saturation time headway

$$1 - \frac{n t_l}{c} = \frac{V_c}{s}$$

① को ~~3600~~ h से devide kiya फिर s से devide किया !

$$\boxed{c = \frac{n t_l}{1 - V_c/s}}$$

if losses and saturation time headway of phases are not same

$$3600 - \frac{3600}{c}\sum_{i=1}^{n} t_{li} = \sum_{i=1}^{n} V_{ci} h_i$$

$$1 - \frac{\sum t_{li}}{c} = \sum_{i=1}^{n} \frac{V_{ci}}{s_i}$$

$$C = \frac{\sum_{i=1}^{n} t_{Li}}{1 - \sum_{i=1}^{n} \frac{V_{ci}}{S_i}}$$

$C \rightarrow$ cycle time in sec.
$n \rightarrow$ no. of phases
$t_{Li} \rightarrow$ loss in i^{th} phase
$V_{ci} \rightarrow$ critical lane vol^m of i^{th} phase
$S_i \rightarrow$ saturation flow / lane for i^{th} phase $\left(\frac{3600}{h_i}\right)$
$h_i \rightarrow$ saturation time headway for i^{th} phase

if $\sum \frac{V_{ci}}{S_i}$ comes ≥ 1 then no. of lanes must be increase so that V_{ci} decreases

no. of lanes is so selected that the value of C is neither too small nor too large.
Generally we try to keep cycle time around 50-90 sec. in multiples of 5

Que. Cycle time of an intersection is 60 sec., Green time is 27 sec, Amber time = 4 sec. saturation headway is 2.4 sec/veh. startup loss time = 2 sec. and clearance loss time = 1 sec. Find the capacity of movement / lane.

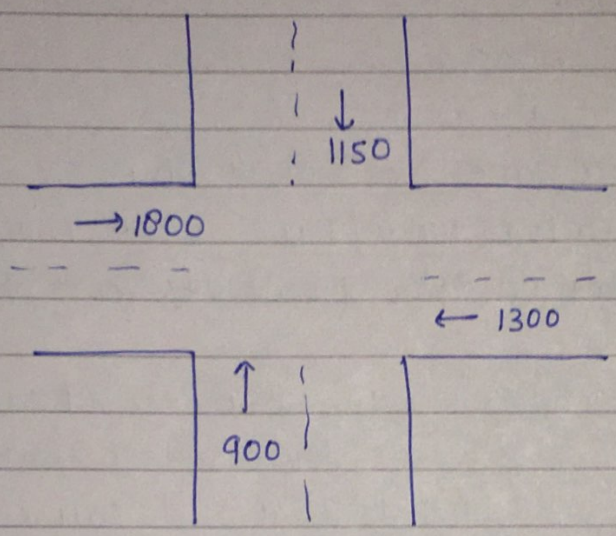
$$\text{capacity} = \frac{3600}{h} \times \frac{g}{c}$$

$$g = G + A - (\text{startup} + \text{clearance})$$
$$= 27 + 4 - (2+1)$$
$$= 28 \text{ sec.}$$

$$\text{capacity} = \frac{3600}{2.4} \times \frac{28}{60} = 700 \text{ veh/hr.}$$

Scanned with CamScanner

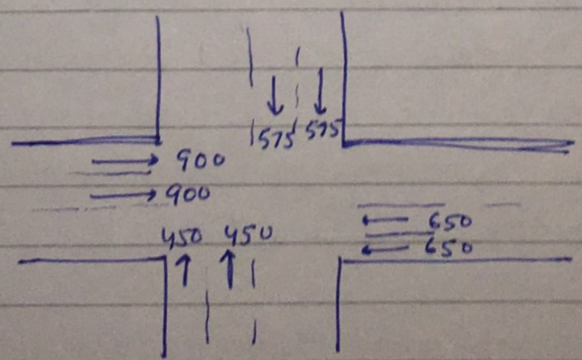
Que: Calculate cycle time for a two phase signal system for the intersection shown below:-
start up loss = 3 sec.
saturation time headway = 2.3 sec.



$V_{cI} = 1150$, $V_{cII} = 1800$

$$\sum_{i=1}^{2} \frac{V_{ci}}{S_i} = \frac{1150}{3600/2.3} + \frac{1800}{3600/2.3} = 1.88 \text{ (Not acceptable)}$$

So, increase the no. of lanes.



$V_{cI} = 575$, $V_{cII} = 900$

$$\sum_{i=1}^{2} \frac{V_{ci}}{S_i} = \frac{575}{3600/2.3} + \frac{900}{3600/2.3} = 0.9423$$

Scanned with CamScanner

$$C = \frac{\Sigma t_{Li}}{1 - \Sigma \frac{V_{ci}}{S_i}} = \frac{2 \times 3}{1 - 0.9423} = 104.11 \text{ sec.}$$

Adopt $C = 105$ sec.

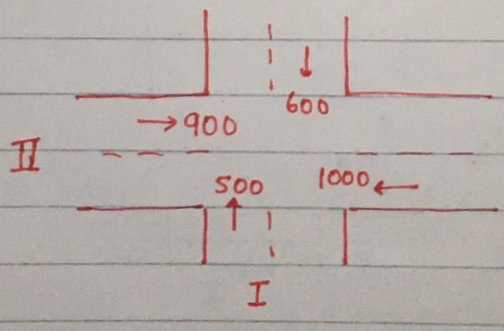
GREEN splitting:- gt is the proportioning of effective Green time of a cycle in corresponding phases. gt is done in proportion of critical lane volume

Eff. Green time in 1 cycle = cycle time − loss in 1 cycle

$$= C - \sum_{1}^{n} t_{Li}$$

Eff. Green of i^{th} phase $g_i = \left(C - \sum_{1}^{n} t_{Li}\right) \frac{V_{ci}}{\sum_{i=1}^{n} V_{ci}}$

Que. phase diagram for flow values at an intersection with 2 phase signal system is shown below. For the loss time and amber time as indicated. Find the green time to each phase. cycle length = 120 sec.



	Ph I	ph. II
loss (s)	3.5 sec	2.5 sec.
Amber (s)	4 sec.	3 sec.

eff. Green time for 1 cycle = 120 − (3.5 + 2.5) = 114 sec.

eff. Green time for phase I = $114 \times \frac{600}{1000} = 42.75$ sec.

$g_I = 42.75$ sec.

$g_{II} = 71.25$ sec.

Scanned with CamScanner

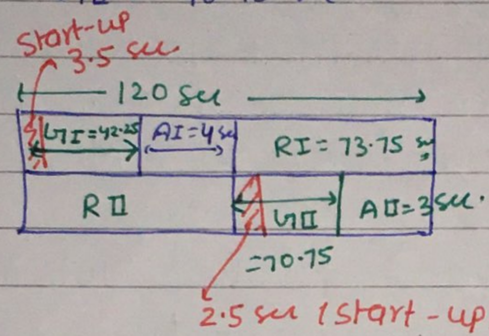
$g_I = G_I + A_I - t_{LI}$

$42.75 = G_I + 4 - 3.5$

$G_I = 42.25$ sec.

$g_{II} = G_{II} + A_{II} - t_{LII}$

$G_{II} = 70.75$ sec.



pedestrain crossing Requirement :-

pedestrain crossing can be taken care in two ways

1) While the signal is Red on a Road, pedestrain Movement can be allowed on that Road.

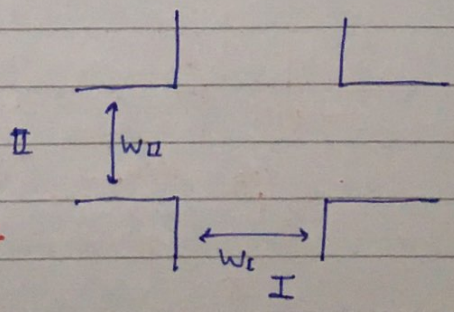
2) By providing exclusive pedestrain phase but it will lead to increase in cycle time

Green time required for pedestrain crossing →

$G_{PI} = 7 + \dfrac{W_I}{1.2}$

walk time (7) ; clearance time ($\dfrac{W}{1.2}$)

$G_{PII} = 7 + \dfrac{W_{II}}{1.2}$



1.2 → pedestrain walking speed (m/sec)

W_I, W_{II} → width of roads I & II (in m)

7 sec. is the Min. Reccommended walk time as per IRC

G_{PI}, G_{PII} → Green times req. for pedestrain crossing on roads I & II respectively.

$$G_{PI} = R_I = G_{II} + A_{II}$$

$$G_{II} = G_{PI} - A_{II}$$

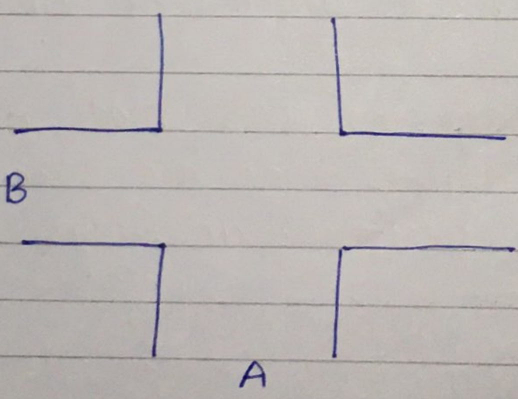
similarly $G_I = G_{PII} - A_I$

Design Methods for traffic signals.

various Methods are –

1) Trial cycle Method
2) Approximate Method
3) Websters Method
4) IRC Method.

① **Trial cycle Method** :- 15 Min. Traffic count n_A & n_B on roads A & B (per lane) during design peak hour flow is counted.



cycle time c is assumed

no. of veh. arriving in one cycle on roads A & B are given as

$$x_A = \frac{n_A C}{15 \times 60}$$

$$x_B = \frac{n_B \times C}{15 \times 60}$$

Green times for phase A & B is calculated as

$G_A = x_A h$

$G_B = x_B h$ h → saturation time headway assume (2.5 sec if not given)

Finally the cycle time is again calculated by adding Green times & Amber times of Both phases.

$$C_f = G_I + A_I + G_{II} + A_{II}$$

$$C_f = \frac{n_A c}{15 \times 60} h + A_I + \frac{n_B c}{15 \times 60} h + A_{II}$$

if the calculated cycle time is = the initially assumed cycle time it means that our assumption was correct and if it is not equal then repeat the process. assuming a New cycle time till the calculated cycle time becomes = Assumed cycle time

Que. 15 Min. Traffic count on cross roads A & B are 178 & 142 Veh/lane. Amber times are 3 sec & 2 secs. For phase A & B. Design cycle timing by Trial cycle Method.

$n_A = 178$, $n_B = 142$

Assume $h = 2.5$ sec.

Assume $C = 45$ sec.

$$x_A = \frac{n_A c}{15 \times 60} = 8.9$$

$$x_B = \frac{n_B c}{15 \times 60} = 7.1$$

Green time are

$G_A = x_A h = 22.25$ sec

$G_B = x_B h = 17.75$ sec

Final cycle time

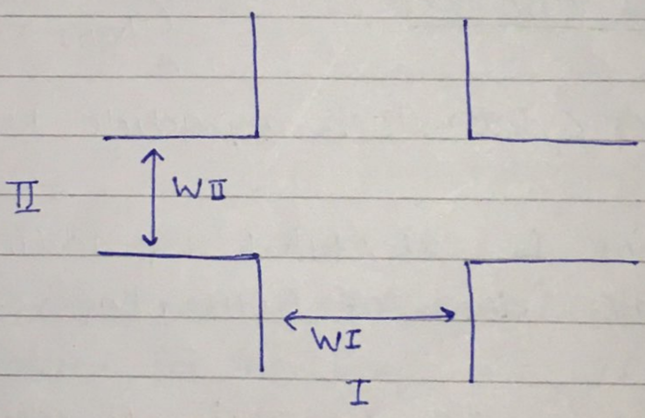
$$C_f = x_A h + A_A + x_B h + A_B$$
$$= 45 \text{ sec.}$$

$C_f = C$ (assumed) Hence correct

cycle time $= 45$ sec.

② APPROXIMATE METHOD :-

In this Method green time is first calculated on the Basis of Green time required for pedestrain crossing and then it is checked for vehicle clearing time.



$$G_{PI} = 7 + \frac{W_I}{1.2}$$

$$G_{PII} = 7 + \frac{W_{II}}{1.2}$$

Green time for Vehicle ⇒

$$G_I = G_{PII} - A_I = G_{PII} - A_I$$
$$G_{II} = G_{PI} - A_{II} = G_{PI} - A_{II}$$

Check for vehicle clearance

$$\frac{G_{veh\,I}}{G_{veh\,II}} = \frac{V_{cI}}{V_{cII}}$$

Where $G_{veh\,I}$ and $G_{veh\,II}$ are Green time for vehicles of phase I and II

V_{cI} and V_{cII} are critical lane Volumes for phase I & II

- Assume $G_{veh\,I} = G_I$ and calculate $G_{veh\,II}$
- If $G_{veh\,II} > G_{II}$ adopt ($G_{veh\,II}$ and G_I)

$$G_{veh\,II} = \frac{G_I \times V_{cII}}{V_{cI}}$$

and if $G_{veh\,II} < G_{II}$ then

put $G_{veh\,II} = G_{II}$ and calculate $G_{veh\,I}$

$G_{veh\,I}$ will always come greater than G_I

And in this case the acceptable Green times will be ($G_{veh\,I}$ and G_{II})

$$G_{veh\,I} = \frac{G_{II} V_{cI}}{V_{cII}}$$

if $G_{veh\,I}$ comes $< G_I$ → it is impracticle . !

Finally cycle time is calculated by adding Green times and Amber times of Both phases

Que. Design cycle time using Approximate Method for the Data Given below.

	I	II
critical lane volume	350	260
Amber time	5 sec	5 sec.
Width (m)	21	15

$$G_{PI} = 7 + \frac{W_I}{1.2} = 7 + \frac{21}{1.2} = 24.5$$

$$G_{PII} = 7 + \frac{W_{II}}{1.2} = 7 + \frac{15}{1.2} = 19.5$$

Check $\frac{G_{veh I}}{G_{veh II}} = \frac{V_{cI}}{V_{cII}}$ =)

$G_{II} = 24.5 - 5 = 19.5$

$G_{I} = 19.5 - 5 = 14.5$

putting $G_{veh I} = G_I$

$G_{veh II} = \frac{G_I \times V_{cII}}{V_{cI}} = \frac{14.5 \times 260}{350} = 10.7$ sec.

$G_{veh II} < G_{II}$ so, Not acceptable

$\frac{G_{veh I}}{19.5} = \frac{350}{260}$ =) $G_{veh I} = 26.25$ sec. $> G_I$

Acceptable

Green times are taken as

$G_I = 26.25$ sec.

$G_{II} = 19.5$ sec.

$C = G_I + A_I + G_{II} + A_{II}$

$= 55.75$ sec.

Adopt $C = 60$ sec

The additional cycle time = ($\Delta C = 4.25$ sec) will be distributed in the Green time of both phases in proportion to their critical Lane volume

$\Delta G_I = 4.25 \times \frac{350}{350+260} = 2.44$ sec.

$\Delta G_{II} = 1.82$ sec.

Final Green times are $G_I = 26.25 + 2.44 = 28.69 \cong 29$ sec.

$G_{II} = 19.5 + 1.82 = 21.32 \cong 21$ sec

Vehicle	Ph I	$G_I = 29$ sec	$A_I = 5$	$R_I = 26$	
	Ph II	$R_{II} = 34$ sec		$G_{II} = 21$ sec	$A_{II} = 5$
Pedestrain	Ph III	Don't Walk		W.T = 8.5	C.T = W.T/1.2 = 17.5
	Ph IV	W.T = 21.5 sec	C.T = W.T/1.2 = 12.5 sec.	Don't Walk	

③ WEBSTER'S METHOD :-

In this Method cycle time is calculated from least total Delay at signalised intersection. It is the **Most** Rational Method. As per this Method the cycle time is given as —

$$C = \frac{1.5L + 5}{1 - Y}$$

$C \rightarrow$ cycle time in sec.

$L \rightarrow$ Total loss in one cycle

$$L = nl + R$$

$n \rightarrow$ no. of phases

$l \rightarrow$ Start-up loss (Assume 2 sec. if Not given)

$R \rightarrow$ All Red loss

$Y \rightarrow$ critical Flow Ratio

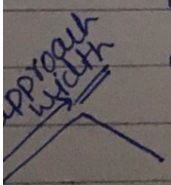
$$Y = \sum_{i=1}^{n} \frac{V_{ci}}{S_i}$$

Where $V_{ci} \rightarrow$ critical lane volume of i^{th} phase

$S_i \rightarrow$ saturation Flow / lane for i^{th} phase $\left(\frac{3600}{h_i}\right)$

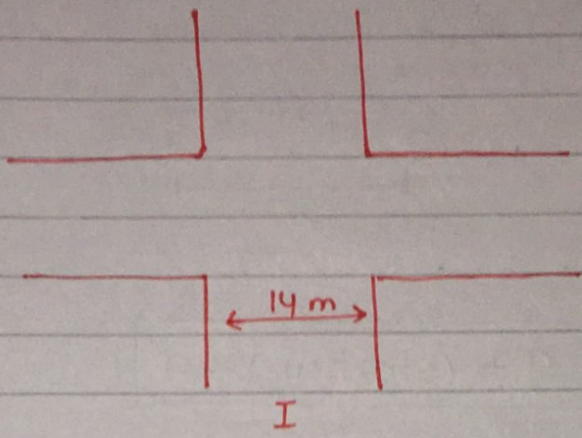
$h_i \rightarrow$ saturation time headway for i^{th} phase (Assume 2.5 sec if not given)

IRC has given the saturation flow for full approach width of carriage way depending on the approach width



Approach width	3	3.5	4	4.5	5	5.5	>5.5
saturation flow for Full approach width {veh/hr}	1850	1890	1950	2250	2350	2990	525 per m. of approach width

Que: Calculate saturation Flow per lane for phase I



per lane → $\frac{1}{2} \times \frac{14}{2} \times 525 = \frac{3675}{2}$ veh/hr. = 1837.5 veh/hr.

After calculating the cycle time effective Green time in one cycle is calculated

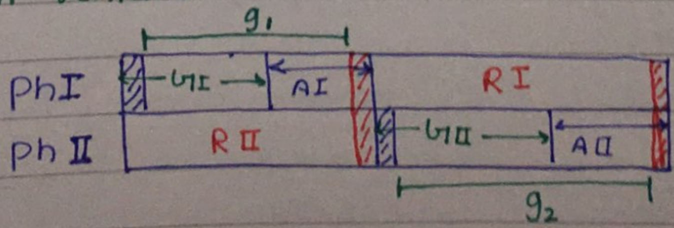
Eff Green time in 1 cycle = C − L

Effective Green time for i^{th} phase is calculated as

$$g_i = (C-L) \frac{\left(\frac{V_{ci}}{S_i}\right)}{\sum\left(\frac{V_{ci}}{S_i}\right)}$$

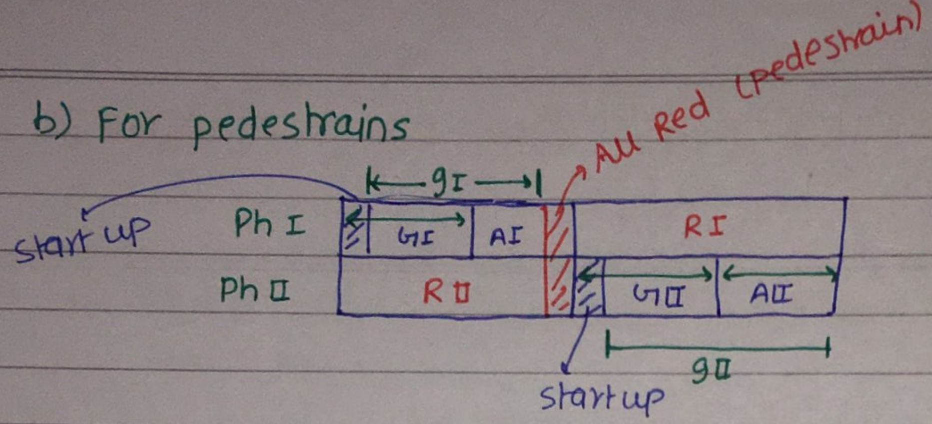
Types of All RED :-

A) For Vehicles :-



$$G = g + (\text{start up} + \text{clearance}) - A$$
(All red)

b) For pedestrains



$$G = g + (\text{startup}) - A$$

Que: Design a two phase signal system for the Data given below using Webster's Method.

q_I = 400 Pcu/hr

q_{II} = 250 Pcu/hr

S_I = 1250 pcu/hr

S_{II} = 1000 pcu/hr

All red for pedestrain = 12 sec.

2 phases ⇒ $n = 2$

$$Y = \frac{V_{cI}}{S_I} + \frac{V_{cII}}{S_{II}}$$

$$= \frac{400}{1250} + \frac{250}{1000}$$

$$= 0.32 + 0.25$$

$$= 0.57$$

loss $L = 2 \times 2 + 12$ Assuming start up loss = 2 sec per phase

$L = 16$ sec.

$$C = \frac{1.5L + 5}{1 - Y} = \frac{1.5 \times 16 + 5}{1 - 0.57} = 67.44 \text{ sec.}$$

Adopt $C = 68$ sec.

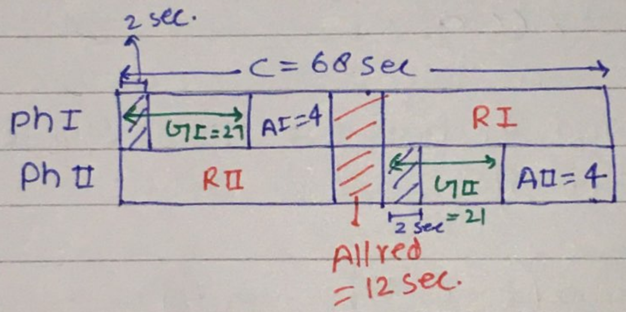
Eff. Green in 1 cycle $= C - L$
$= 68 - 16 = 52$ sec.

Eff. green for phase 1 $= \frac{52 \times 0.32}{0.57} = 29.19 = 29$ sec

Eff. Green for phase 2 $= 52 - 29 = 23$ sec.

$G_I = g_I + (\text{Start-up})_I - \text{Amber}_I = 29 + 2 - 4 = 27$ sec
$G_{II} = g_{II} + (\text{Start-up})_{II} - \text{Amber}_{II} = 23 + 2 - 4 = 21$ sec.

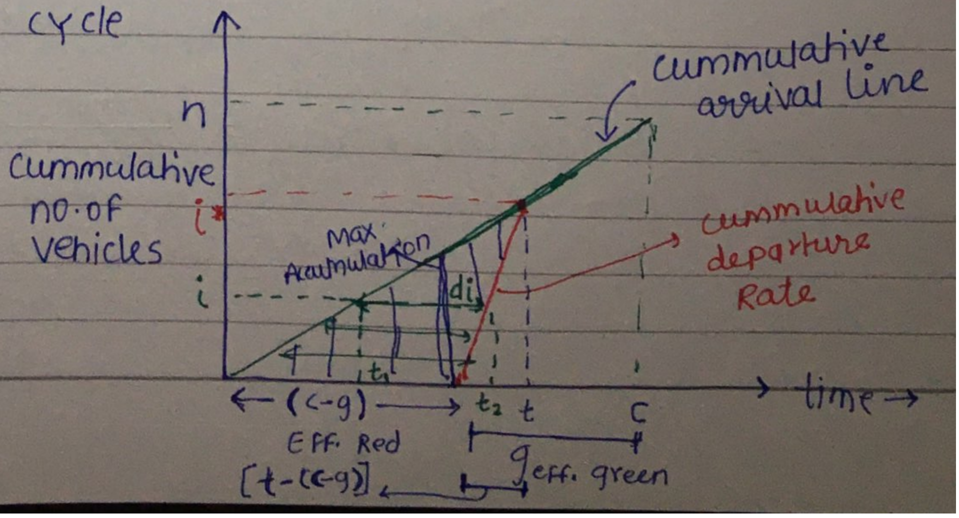
Assuming Amber = 4 sec/phase



DELAY ANALYSIS :-

Assumptions :-

1) Arrival process is deterministic and veh. arrive at a uniform rate
2) System is unsaturated (That is total no. of vehicles arriving in a period is less than total no. of vehicles that can be served by the system.
It implies that veh. arriving in a cycle are cleared in the same cycle



C → is the cycle time

g → eff. Green time

d_i → delay for the i^{th} vehicle $(t_2 - t_1)$

V → slop of cummulative arrival line i.e Uniform Rate of arrival.

S → slop of cummulative Departure line i.e saturation Flow rate.

Assuming No. of vehicles to be large we've Total delay

$$\text{Total Delay} = \frac{1}{2} \times (c-g) \times i^* \qquad (\simeq \Sigma d_i$$

No. of vehicles arriving in time t = No. of vehicles cleared in time $[t-(c-g)]$

$$Vt = S\left[t-(c-g)\right] \quad \text{---(i)}$$

$$t = \frac{S(c-g)}{S-V}$$

$$i^* = Vt = \frac{SV(c-g)}{S-V} \quad \text{---(ii)}$$

From eqⁿ ① & ②

$$\text{Total Delay} = \frac{1}{2} \times (c-g)^2 \cdot \frac{SV}{S-V} = \frac{SV(c-g)^2}{2(S-V)}$$

$$\text{Avg. Delay per Vehicle} = \frac{\text{Total Delay}}{\text{Total no. of Vehicles in one cycle}}$$

$$= \frac{SV(c-g)^2}{2(S-V) \times VC}$$

$$\text{Avg. Delay per Vehicle} = \frac{c(1-g/c)^2}{2(1-v/s)}$$

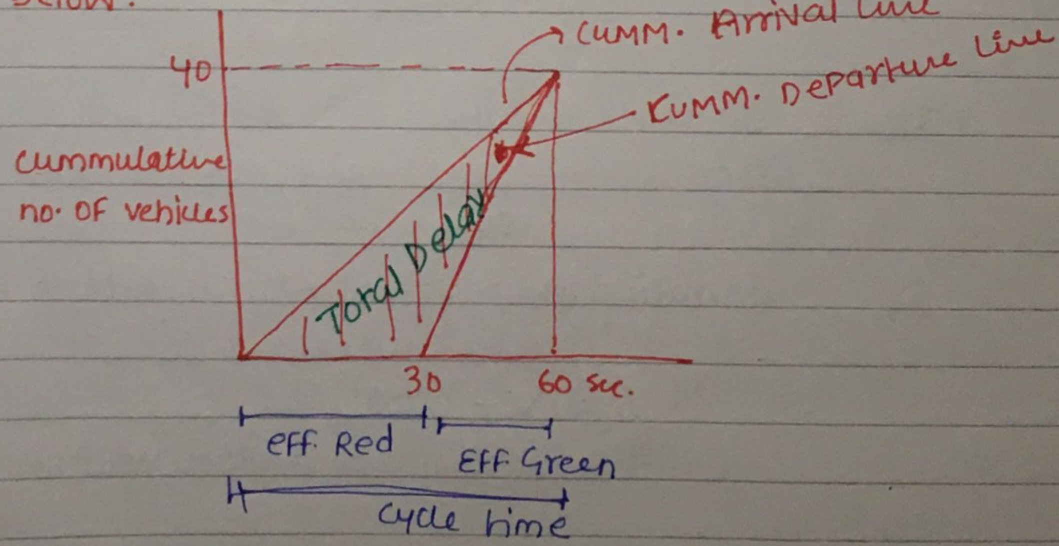
① Horizontal ordinate b/w cummulative arrival and cummulative departure line represents delay.

② Vertical ordinate between cummulative arrival and cummulative departure line represents cummulation or queue.

③ Area of Triangle b/w cummulative arrival and cummulative departure line = Total delay of all veh.

Saturation Flow Rate = Arrival Rate + Rate of decrease of queue

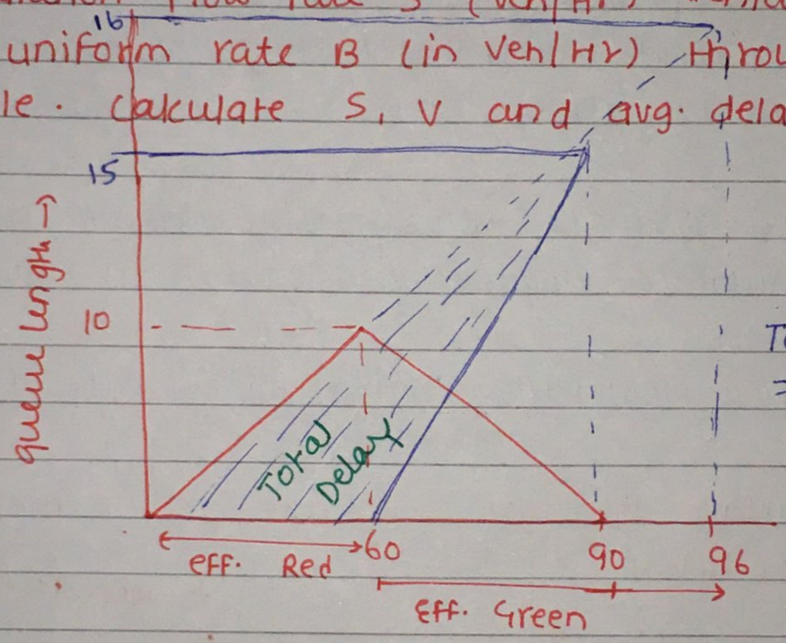
Que. calculate the avg. delay per vehicle for the Figure shown below.



$$\text{Avg. delay per veh} = \frac{\frac{1}{2} \times 30 \times 40}{40}$$

$$= 15 \text{ sec.}$$

Que. The queue length (in no. of vehicles) vs time (sec) plot for an approach to a signalized intersection with the cycle length of 96 sec. is shown in the figure below (not drawn to scale) at time t=0 the light has just turned red. The effective green time is 36 sec. during which vehicle discharge at saturation flow rate S (Veh/Hr). Vehicles arrive at a uniform rate B (in Veh/Hr) throughout the cycle. Calculate S, V and avg. delay per vehicle.



Total delay
$= \frac{\frac{1}{2} \times 60 \times 15}{16}$

V → 60 sec —— 10

3600 —— $\frac{10}{60} \times 3600 = 600$ Veh/hr.

S → saturation flow rate = Arrival rate + rate of decrease of queue

$S = V + 1200$

$S = 600 + 1200 = 1800$ veh/hr.

Avg. delay per Veh. $= \dfrac{\frac{C}{2}(1 - g/c)^2}{(1 - V/S)}$

$= 28.125$ sec.

Peak Hourly Factor (PHF) :-

It is used to represent variation in Hourly traffic. It is defined as the Ratio of 60 Min. Volume in peak hour to 4 times peak 15 Min. Volume

$$(PHF)_{15} = \frac{V_{60}}{4 \times V_{15}}$$

V_{60} → 60 Min Vol^m in peak hr.

V_{15} → peak 15 Min vol^m in that hour.

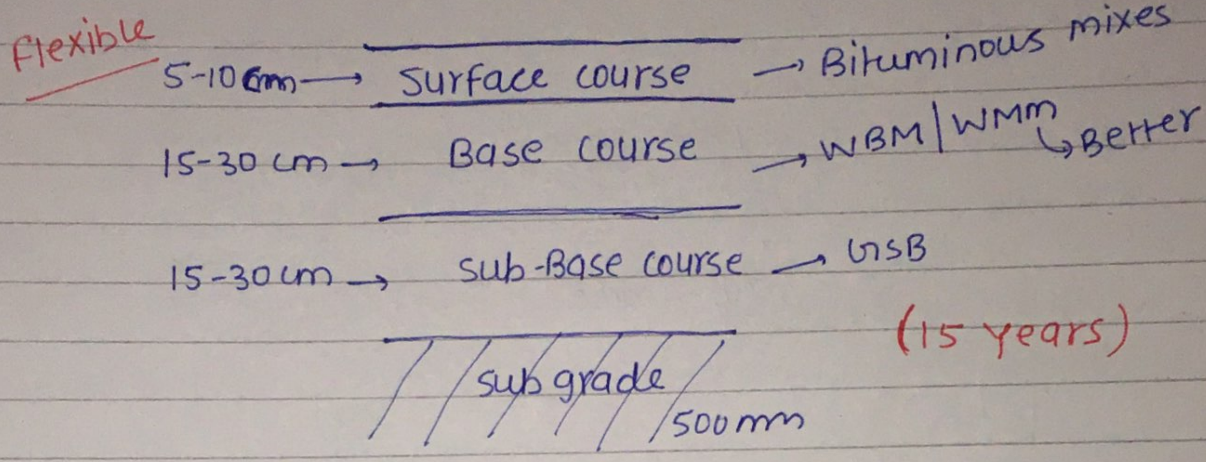
$$0.25 \leq (PHF)_{15} \leq 1$$

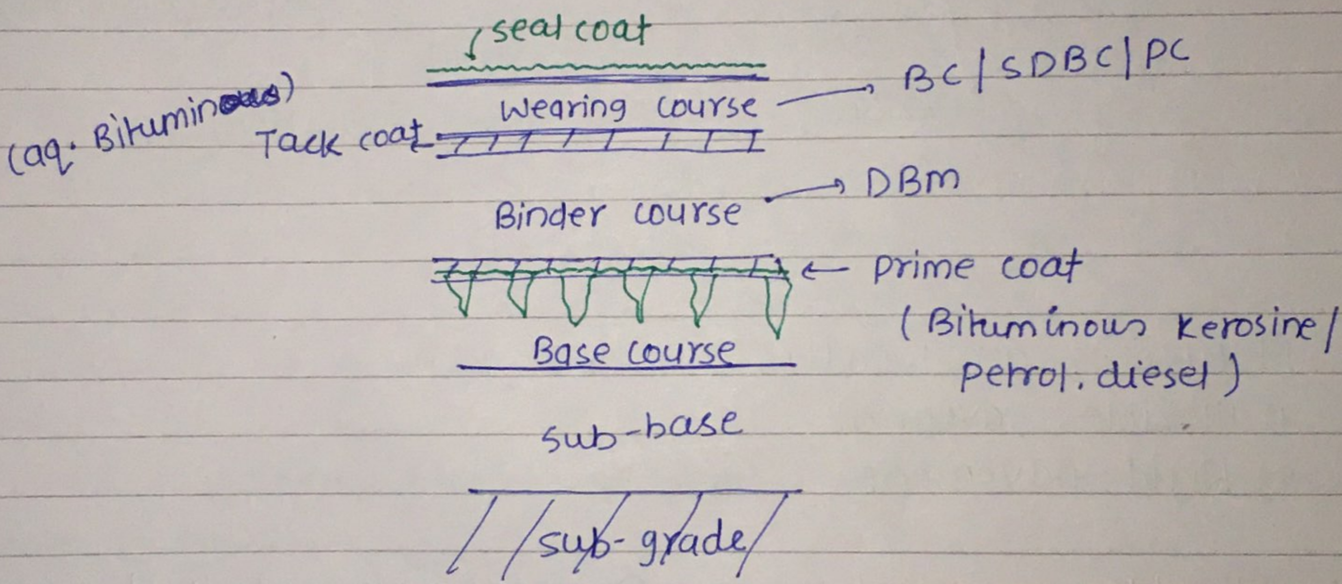
1 → incase of uniform Arriva

0.25 → in case of Max. variation

$$(PHF)_{20} = \frac{V_{60}}{3 \times V_{20}} \rightarrow 0.33 \leq (PHF)_{20} \leq 1$$

PAVEMENT-DESIGN





SDBC → semi dense Bituminous concrete
PC → premix carpet
BC → Bituminous concrete
Wmm → wet mix Macadem

Seal coat → improves impermeability and skid resistance

Tack coat → provides Bonding b/w two layer

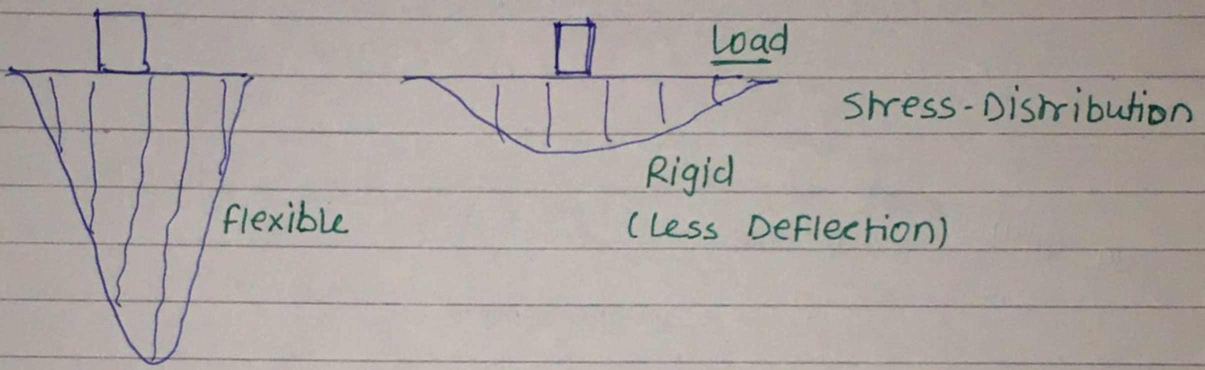
Prime coat → plugs voids of Base course and prepares the surface for application of track coat.

Rigid (30 years)

15-30 cm → Surface course → cement concrete

15-30 cm → Base/sub-Base course → optional (DLC) Dry ~~link~~ Lean concrete

sub-grade



Pavements are broadly classified as –

1) Flexible pavement
2) Rigid pavement

1) Flexible pavement :- In flexible pavement load from the Top layer is distributed to the bottom layer through grain to grain contact. We need to use better quality material in the Top layers as compare to the bottom layers because the stresses in the bottom layers will be lower than the Top. The pavement layer thickness is decided in such a way that stress in each layer does not exceeds it's permissible value.
In flexible pavements any deformation in the bottom layers is reflected on the top layers.

purpose of different layers in flexible pavements are

a) subgrade → This layer is beneath the sub-base layer. It is prepared from natural soil by compacting it to 95-98% of proctor density. Generally 500 mm is compacted (300 mm for Rural low volume Roads). It is designed to recieve stresses from the upper layers such that the verticle compressive stress does not exceeds it's permissible value.

b) Sub-Base :- This layer is beneath the Base course and it's primary function is to provide drainage and structural support to the upper pavement layers. It also reduces intrusion of fines into the pavement layers.

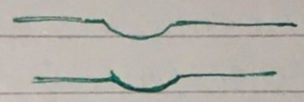
High quality sub-Grade with steep slop may not require sub-Base.

c) Base-course :- It is immidiatly below the surface course. It provides structural support by bearing high stresses coming from the top layers and distributes them to the lower layers. It also contributes to sub-surface Drainage.

d) surface-course :- It is also known as wearing course. It is of highest quality and Generally Bituminous mixes are used. It provides an overall smooth surface, skid resistance for tyres and sustains environmental and weathering actions.

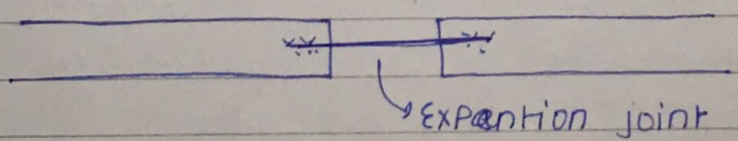
Failure of Flexible pavement :- Major Modes of Failure of Flexible pavements are:-

1) Fatigue cracking (crocodile cracking | Eligator cracking)
2) Thermal cracking
3) Rutting



However IRC considers only fatigue cracking and Rutting due to sub-grade deformation. To control fatigue cracking We limit the tensile strain at the Bottom of surface course and to control Rutting the We limit the axial compressive strain at the sub-Grade layer

2) Rigid pavements :- In this case load is distributed due to Flexural action of the slab. (cement - concrete). The purpose of Base / sub-Base course in Rigid pavement is to prevent Mud-pumping, provide Drainage and reduce deflections



Base / sub-Base course is optional in Rigid pavement.

Failure of Rigid pavement :-

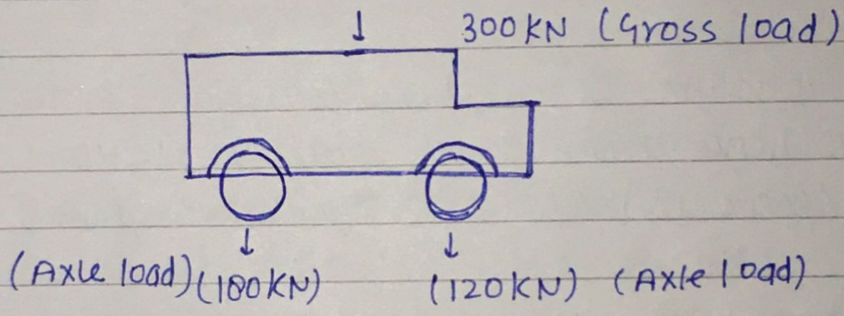
Modes of Failures are -

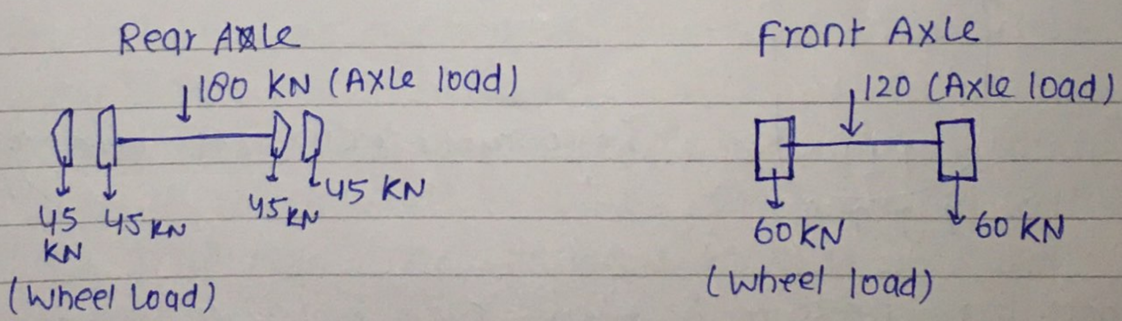
1) fatigue cracking
2) Thermal cracking
3) Mud pumping

IRC gives design steps only for Fatigue cracking and not for Mud - pumping

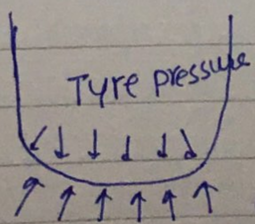
Load and Traffic Basic concept :-

For Geometric design we need to consider all types of vehicle but for pavement design only vehicles having significantly heavy load are considered.
These vehicles are generally commercial vehicles. As per IRC veh. having Gross load > 3 Ton (≥) are called commercial veh. and only these vehicles will be considered in design

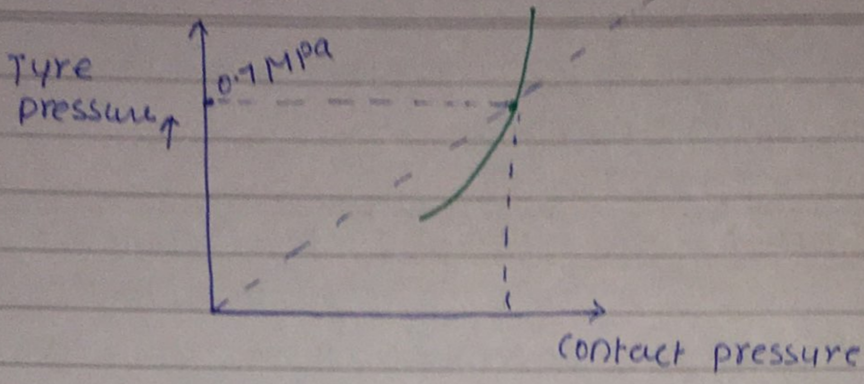




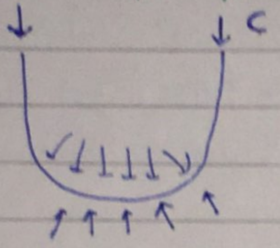
TYRE pressure and contact pressure :-



$$\text{Contact pressure} = \frac{\text{Wheel load}}{\text{Contact Area}}$$



Case-I When tyre pressure is low (< 0.7 MPa)

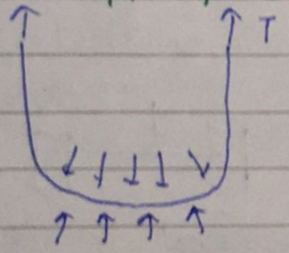


$$\text{contact pressure} \times A = \text{Tyre pressure} \times A + C$$

When tyre pressure is low Tyre Materials comes under compression hence from force Balance we've

contact pressure > Tyre pressure

Case-II When tyre pressure is high (> 0.7 MPa)



$$T + \text{contact pressure} \times A = \text{tyre pressure} \times A$$

When type pressure is high, Tyre Materials comes under Tension. Hence from force Balance we've

contact pressure < Tyre pressure

$$\text{Rigidity Factor} = \frac{\text{Contact pressure}}{\text{Tyre pressure}}$$

a) When Tyre pressure < 0.7 MPa

$R.F > 1$

b) When tyre pressure = 0.7 MPa. R.F = 1
c) When tyre pressure > 0.7 MPa , R.F < 1

NOTE IRC recommends a tyre pressure of 0.8 MPa for design.
Contact Area of the wheel is taken as circular for design.

DESIGN load consideration:- The Following effects are considered while calculating Design load for the pavement -
1) Traffic volume in each year will increase on the Road
2) Wheel loads are applied over different portions of the pavement and not at the same location
3) Different vehicles have different Weight

Related to above mentioned effects various design parameters are
(i) Traffic Forecast :- In this we calculate the total no. of commercials vehicles that are going to utilise the road over the design Life of the Road.

$$N = \frac{365A\left((1+r)^n - 1\right)}{r}$$

$$A = P(1+r)^x$$

N → cummulative no. of commercial vehicles during the Design Life of the Road
A → initial design traffic in vehicles /day in the year of completion of construction

R → Traffic Growth Rate (in decimal)
n → Design life in years of the pavement
P → Traffic per day as per last count
x → planning & construction period in years

NOTE • As per IRC 37 : 2001 Traffic Growth Rate were taken as 7.5% which was later on modified to 5% as per IRC 37 : 2012

• Design life of pavements are taken as

flexible pavements → 20 yrs for expressway / Urban Roads
→ 15 yrs for NH / SH

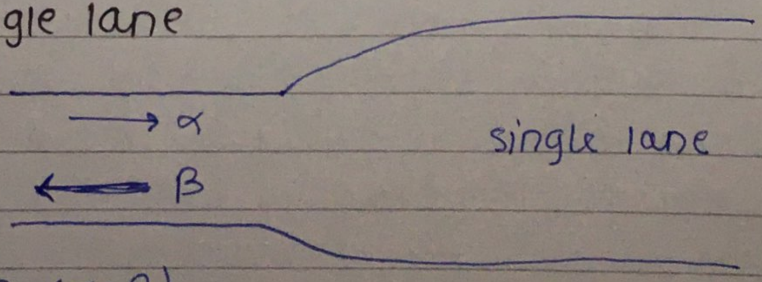
Rigid pavements → 30 yrs

2) Lateral Distribution of Wheel load :- All vehicles do not move on the same path Hence they are not going to load the pavement at the same point. Thus all commercial vehicles counted during the design life (N) should not be taken into account and only a certain % of cummulative traffic should be considered. This is accounted for by taking lateral distribution or lane distribution factor (LDF)

IRC Recomendations for LDF :- (Based on Road to be constructed)

(i) For single carriageway

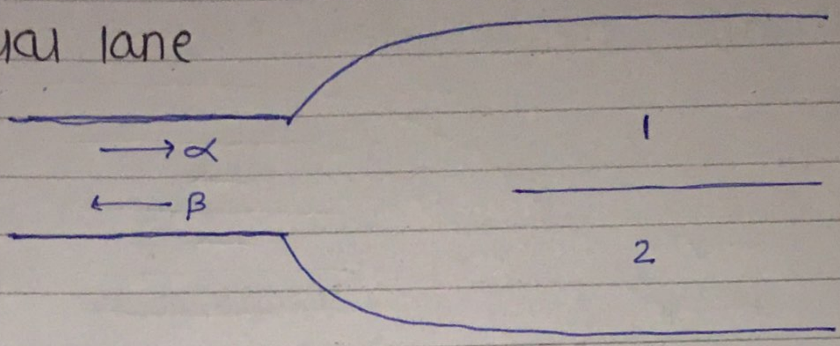
a) single lane



$P = (\alpha + \beta)$

LDF = 1

b) Dual lane



$P = (\alpha + \beta)$

LDF = 0·75 outdated

NOTE IRC : 37 : 2001 Modification

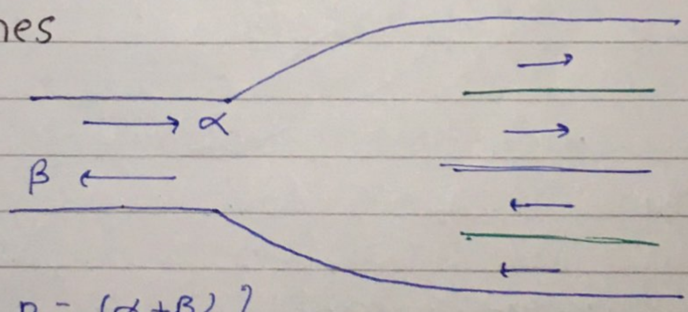
For calculating LDF as per IRC 37: 2012 →

Either 50 % of total traffic is considered { $P = \alpha + \beta$, LDF = 0·5 }

or 100 % of the traffic in the direction of higher VDF

{ $P = \alpha$ or β whichever has higher LDF, LDF = 1 }

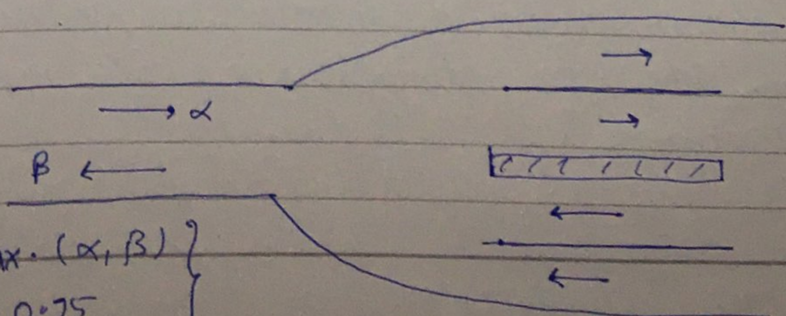
c) 4 - lanes



{ $P = (\alpha + \beta)$, LDF = 0·4 }

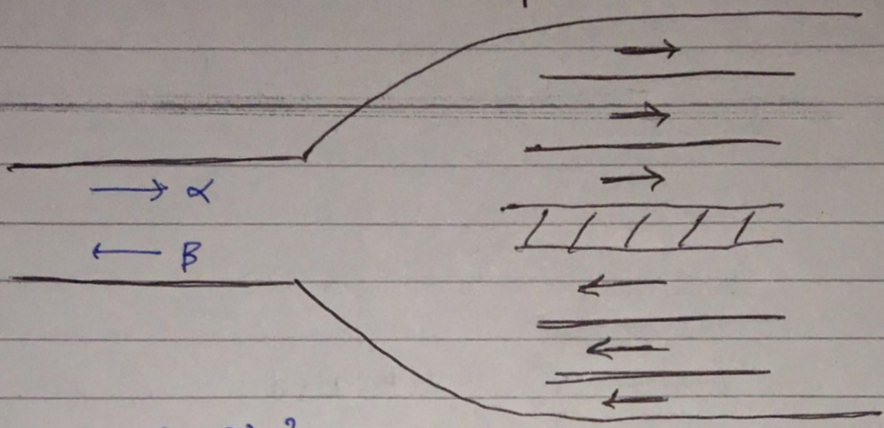
(ii) For Dual carriageway

a) Dual 2 - lane carriage way (i.e 4 lane devided highway)



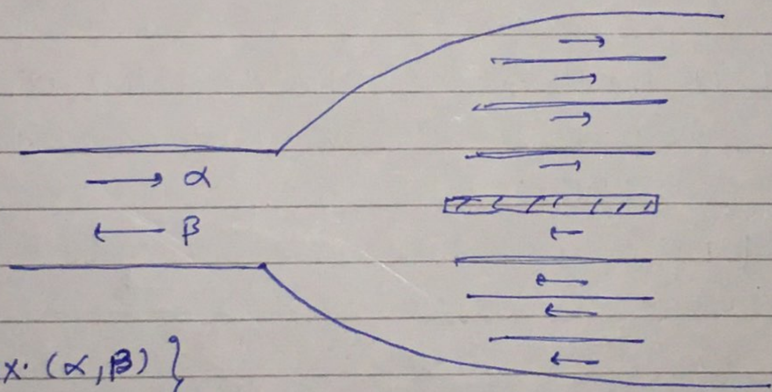
{ $P = \max.(\alpha, \beta)$, LDF = 0·75 }

b) Dual 3-lane carriage way (6-lane devided Highway)



$$\left\{ \begin{array}{l} P = max.(\alpha, \beta) \\ IDF = 0.6 \end{array} \right\}$$

c) Dual 4-lane carriage way (8-lane devided highway)



$$\left\{ \begin{array}{l} P = max.(\alpha, \beta) \\ IDF = 0.45 \end{array} \right\}$$

Design no of vehicles (per lane) in design life $= N \times IDF$

3) Vehicle Damage Factor (VDF) Different commercial vehicles will have diff^n weight. Hence we try to convert the design no. of commercial vehicles during the design life (per lane). into no. of standard Axle repeatitions

This is done by taking VDF. Hence cummulative no. of standard axles (CSA) for design is given as

$$CSA = N \times IDF \times VDF \times 10^{-6} \text{ MSA}$$

* $$CSA = \frac{365A((1+r)^n - 1)}{r} \times IDF \times VDF \times 10^{-6} \text{ msa}$$

Where $VDF = \frac{\text{Total No. of standard Axles}}{\text{No. of veh. surveyed}}$ Std. Axle/veh.

* standard Axle = 80 kN

Equivelent Axle load Factor :- (Fixed vehicle approach)

80 kN single Axle is considered as standard Axle.

Axles that are not 80 kN are converted to standard Axle using equivelent Axle load factor (EALF)

$$EALF = \left(\frac{P_{kN}}{80}\right)^4$$

* Total no. of standard Axle repeatitions is given as $= \sum_{i=1}^{m} n_i f_i$

$m \rightarrow$ no. of class intervals of Axle load

$n_i \rightarrow$ no. of Axles in i^{th} class interval

$f_i \rightarrow$ EALF for i^{th} class interval calculated at the mid point of the class interval

NOTE Total no. of standard Axles when devided by no. of vehicles surveyed will give VDF

Que. The Result of one day Axle load survey of truck on a road Is given below. Find the total no. of repeatitions of standard Axle in one year.

	class interval (KN)	frequency	EALF $(f_i) = (P/80)^4$	$n_i f_i$
20	0–40	50	3.91×10^{-3}	$3.91 \times 10^{-3} \times 50$
60	40–80	250	0.316	0.316×250
100	80–120	400	2.44	2.44×400
140	120–160	250	9.38	9.38×250
		$\Sigma = 950$		$\Sigma = 3400.6$

no. of std. Axle in 1 yr $= 365 \times 3400.6 \times 10^{-6}$

$= 1.24$ msa

Que. In the previous Que. for 950 Axles surveyed No. of vehicles were 400 only. calculate the VDF

$$VDF = \frac{\text{No. of standard Axles in survey}}{\text{No. of vehicles suryed}}$$

$$VDF = \frac{3400.6}{400} = 8.5 \text{ std Axle/Veh.}$$

Que. out of 500 Veh. 200 veh. have a VDF of 3.5 and the remaining veh. have VDF of 2.5. What is the avg. VDF = ?

$$\frac{200 \times 3.5 + 300 \times 2.5}{500} = 2.9 \text{ Std. Axle/veh.}$$

Que. It is proposed to widen and strengthen a two lane NH section into a 4-lane devided highway. The existing traffic in one dir^r is 2500 CVPD. The construction will take one year and the design CBR of soil subgrade is found to be 5%. VDF = 3.5 Std. Axle/CV. Design life = 10 Yrs. and Growth Rate for CV = 8%. calculate cummulative Std. Axle in msA.

$$CSA = \frac{365\,A\left((1+r)^n - 1\right)}{r} \times LDF \times VDF \times 10^{-6}$$

$$A = P(1+r)^x$$

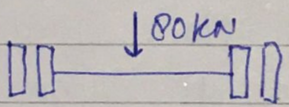
$$A = 2500\,(1.08)^1 = 2700$$

$$CSA = \frac{365 \times 2700\left((1+0.08)^{10} - 1\right)}{0.08} \times 0.75 \times 3.5 \times 10^{-6}$$

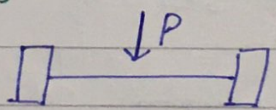
$$CSA = 37.47 \text{ msa}$$

IRC 37:2012 Modification :-

→ single Axle with dual wheel set up on Either sides carrying a load of 80KN is called standard Axle.

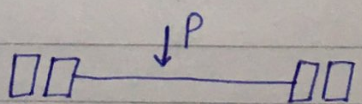


a) EALF For single Axle with single wheel set up on either sides



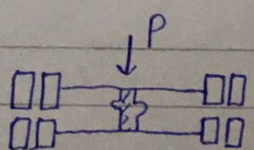
$$= \left(\frac{P}{65}\right)^4$$

b) EALF For single Axle with Dual Wheel set up on either side



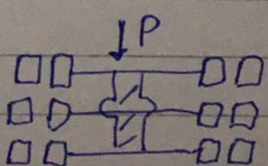
$$= \left(\frac{P}{80}\right)^4$$

c) EALF for tandem Axle



$$= \left(\frac{P}{148}\right)^4$$

d) EALF for Tridem Axle



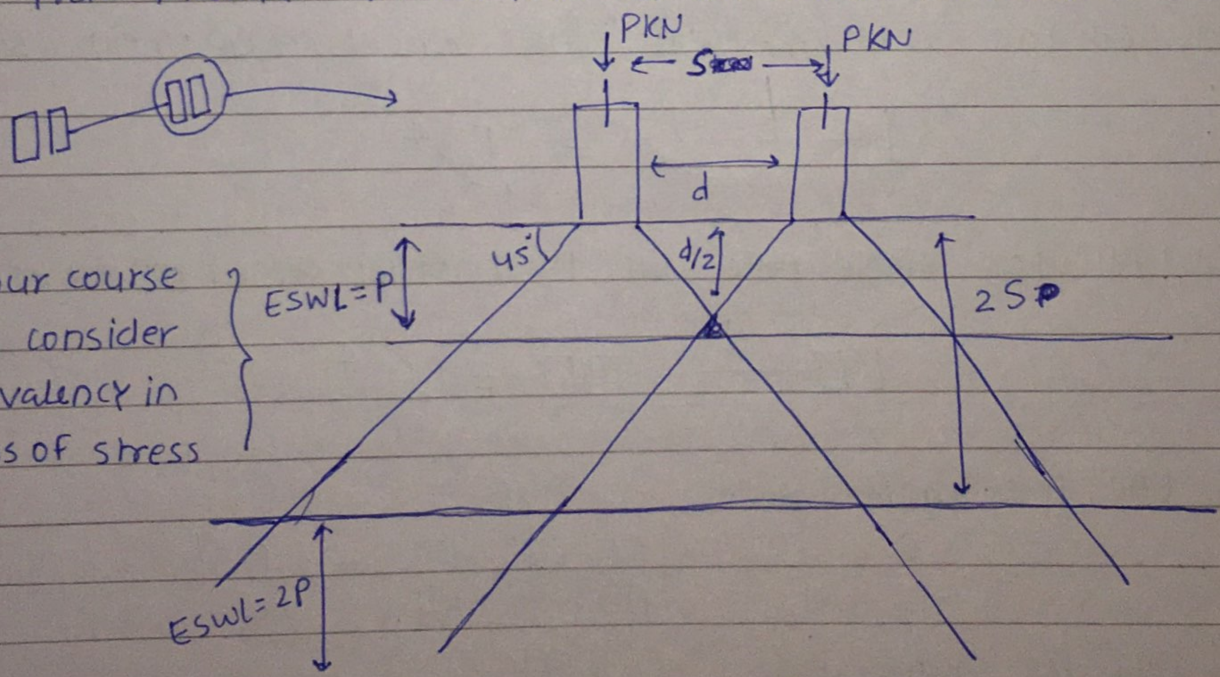
$$= \left(\frac{P}{224}\right)^4$$

Min. sample size to be surveyed

CVPD	Min %
<3000	20
3000-6000	15
>6000	10

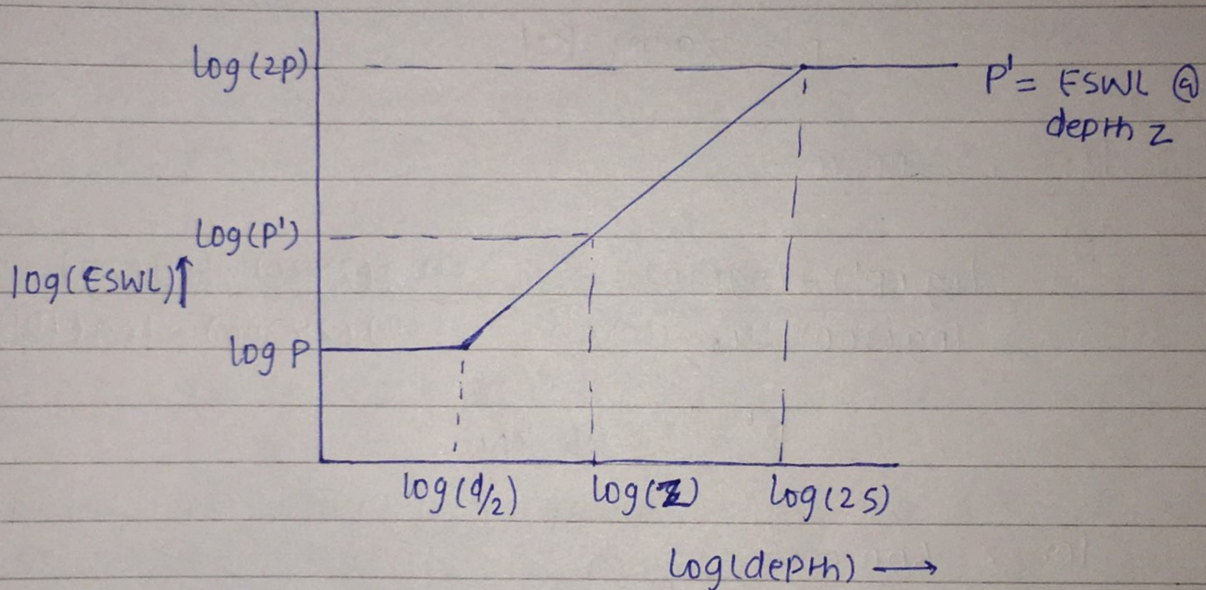
Equivelent single Wheel load :- (Fixed traffic approach)

In this approach we convert a dual wheel set up into equivelent single wheel load for analysis. This approach was used in the design of Airport pavements.
Equivelent single wheel load (ESWL) is defined as the load on single wheel which will cause an equivelent magnitude of selected parameter like stress, strain or deflection etc. at a given location equal to that resulting from multiple wheel load at the same location.



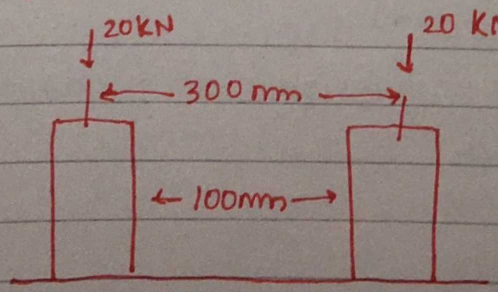
In our course we'll consider equivalency in terms of stress

- Upto depth $d/2$ NO stress overlaping occurs Hence for depth upto $d/2$ ESWl is taken as p.
For depth $\geq 2S$, the effect of overlapping is very large Hence for depths $\geq 2S$ ESWl is taken as $2p$
ESWl for depth b/w $d/2$ and $2S$ is calculated assuming a linear variation b/w load and depth when plotted on a log - log scale



$$\frac{\log(P') - \log(P)}{\log(z) - \log(d/2)} = \frac{\log(2P) - \log(P)}{\log(2S) - \log(d/2)}$$

Que: Calculate ESWl corresponding to depth 200, 300 & 600 mm for the figure shown below



$P = 20$ kN
$S = 300$ mm
$d = 100$ mm

For $z = 200$ mm

$$\frac{\log(P') - \log(20)}{\log(200) - \log(100/2)} = \frac{\log(2\times20) - \log(20)}{\log(2\times300) - \log(100/2)}$$

$$P' = 29.44 \text{ KN}$$

For $z = 300$ mm.

$$\frac{\log(P') - \log(20)}{\log(300) - \log(100/2)} = \frac{\log(2\times20) - \log(20)}{\log(2\times300) - \log(100/2)}$$

$$P' = 32.96 \text{ kN}$$

for $z = 600$ mm

$$P' = 2P = 40 \text{ KN}$$

Design Methods for Flexible pavement, (IRC-37)

various approaches are:-

1) Emperical approach:- Group Index Method
   CBR Method
   California resistance Value Method

2) semi-Emperical Approach (IRC Method):- Tri-Axial Mothod, IRC Method

3) Theortical Method (Mechanistic Aproach):- Burmister Method.

① **Group Index Method:-** Group Index is an arbitory index and it's value varies from 0-20. Higher is the Group Index, weaker is the soil.

Group Index for soil subgrade is expressed as.

$$GI = 0.2a + 0.005ac + 0.01bd$$ Expressed as whole no.

$a = P-35$ } $\ngtr 40$ Expressed as whole no. from 0-40
$b = P-15$

$c = W_L-40$ } $\ngtr 20$ Expressed as whole no. from 0-20
$d = IP-10$

P → % passing through 75 μ sieve
W_L → Liquid Limit
I_P → plasticity index (Liquid Limit − Plastic Limit)

Based on anticipated traffic per day and group index of subgrade thickness of pavement is calculated based on design charts provided by IRC

class	Anticipated traffic (veh/day)
light	<50
Medium	50-300
Heavy	>300

NOTE· Thickness of Sub-Base depends only on Group Index whereas thickness of Base & surface course depends both on ~~subgrade~~ group Index as well as Anticipated traffic

Scanned with CamScanner

Classification of soil Based on GI

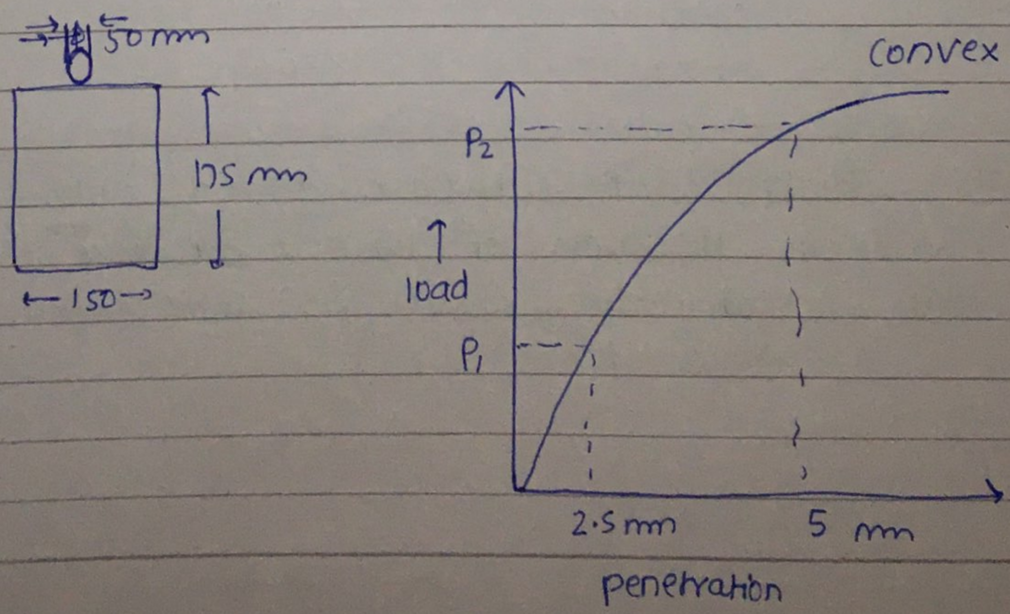
GI	Class
0-1	Good
2-4	Fair
5-9	poor
10-20	Very poor

② California Bearing Ratio Method [CBR]:-

CBR Testing is done to determine material property for pavement design. In this Method a 150 mm dia mould is filled with soil (remoulded) and kept on a Base plate. loading is then applied through a 50 mm dia plunger @ rate of 1.25 mm/Min.

NOTE The sample is soaked for 4 days before testing unless the annual Rainfall is <50 cm in which case soaking is not required.

The corresponding load penetration curve is plotted and from that CBR value is calculated



penetration resistance corresponding to 2.5 mm and 5 mm is noted and from that CBR value is calculated.

$$(CBR)_{2.5mm} = \frac{P_1}{\text{load corresponding to 2.5 mm penetration of standard material}} \times 100$$

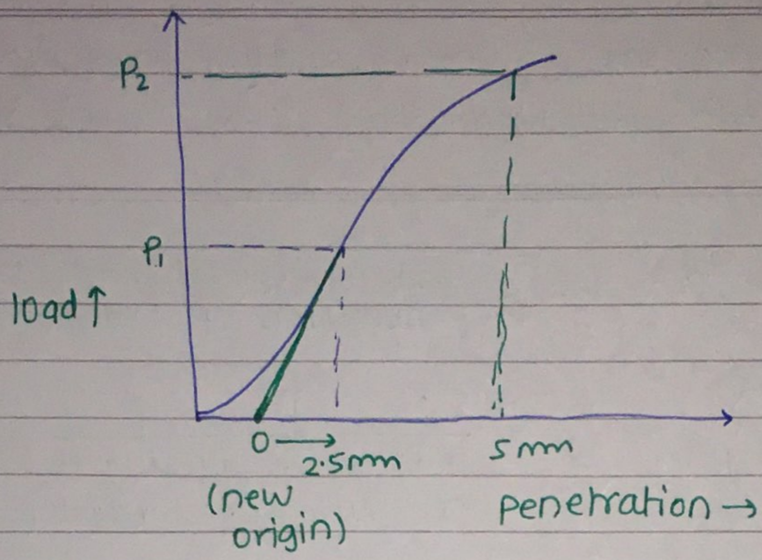
$$(CBR)_{5mm} = \frac{P_2}{\text{load corresponding to 5 mm penetration of standard material}} \times 100$$

load values for standard material (crushed stone) for 2.5 mm penetration is 1370 kg or 70 kg/cm² and for 5 mm penetration it is taken as 2055 kg or 105 kg/cm²

If the CBR corresponding to 2.5 mm comes greater than (CBR) 5 mm then the CBR value of soil is taken as $(CBR)_{2.5 mm}$

if However (CBR) 5 mm comes > $CBR_{2.5 mm}$ we'll repeat the test and if the same result repeats then CBR value of soil is taken as $(CBR)_{5mm}$

If the speciman has surface irregularities or has become slury or the plunger is inclined, the initial portion of load penetration curve May have a concavity upwards. in that case a tangent is drawn from the ~~deepest~~ steepest point on the curve and the point where this tangent cuts the penetration axis is taken as the new origin. load values corresponding to 2.5 & 5 mm penetration is calculated from the new origin.

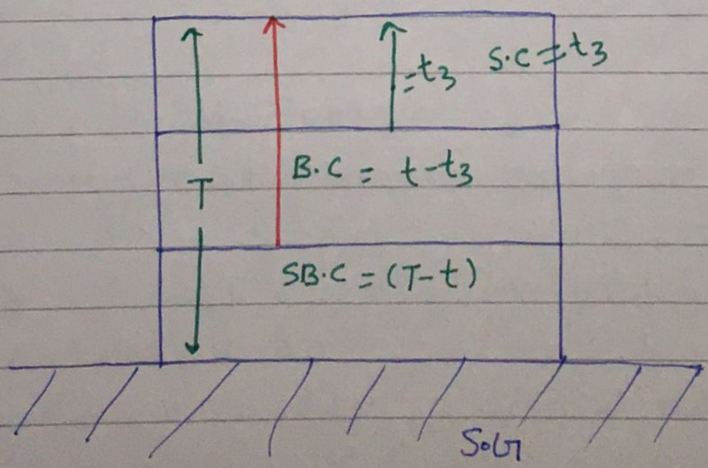


Based on CBR value of a layer, thickness of pavement required above that layer is calculated using the following formula

$$t = \sqrt{\frac{1.75P}{CBR(\%)} - \frac{A}{\pi}}$$

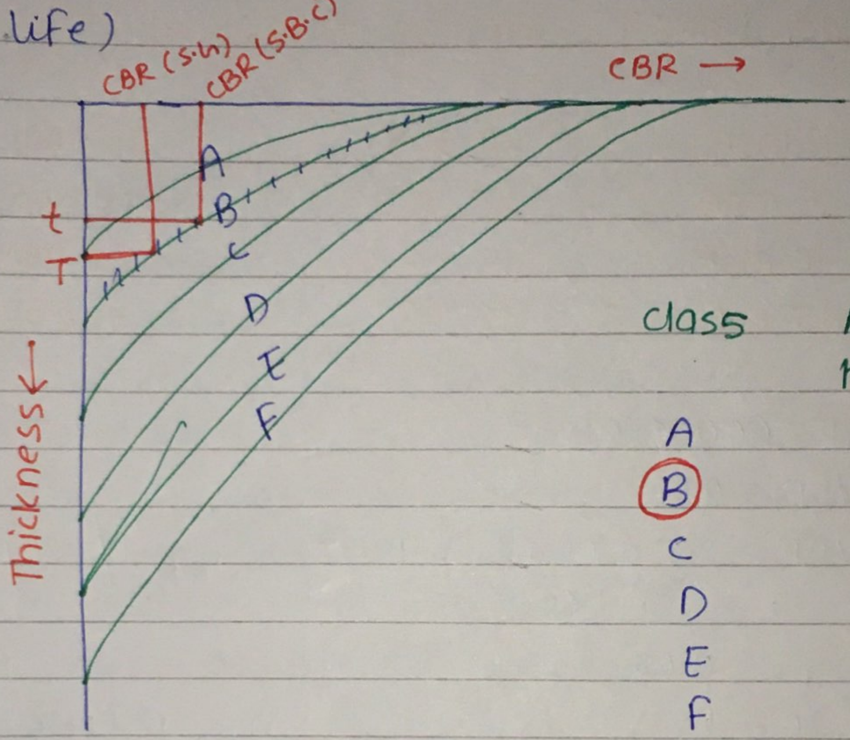
$t \rightarrow$ thickness of pavement (cm) above a layer
$P \rightarrow$ wheel load (kg)
$A \rightarrow$ contact area of wheel (cm^2)

above eq^n is valid only when CBR < 12%.



$T \rightarrow$ thickness req. above S.G
$t \rightarrow$ thickness req. above S.B.C
$t_3 \rightarrow$ thickness req. above B.C

IRC 37:1970 IRC has provided design charts to calculate pavement thickness Based on CBR value and anticipated traffic / day (At the end of Design life)



Class	Anticipated traffic (veh/dy)
A	0-15
B	15-45
C	;
D	;
E	;
F	>4500

$$\text{Anticipated traffic} = P(1+r)^{x+10}$$

P → traffic / day as per last count
r → traffic Growth Rate
x → construction & Planning period in years

IRC 37:2012

IRC has provided design charts for the calculation of pavement thickness of various layers Based on CBR of sub-grade and cummulative no. of standard Axles (CSA)

Que: Design a flexible pavement for 7m wide carriageway out of 600 CVPD, 200 CVPD have a VDF of 2.5 and the remaining veh. have VDF of 3.5. Design the pavement if the eff. CBR of sub grade is 6%. planning and construction period is 1.5 years

Design Life = 15 yrs

Assume suitably data Necessarily.

Design Traffic	Wearing course (mm)	Binder course	Base course	SUB-Base course (GSB)
2 msA	20 SDBC	50 DBM	225 WBM	175
5 msA	25 SDBC	50 DBM	250 WBm	210
8 msA	40 BC	60 DBM	250 WBM	240 -
10 msA	40 BC	65 DBM	250 WBM	260
15 msA	40 BC	90 DBM	250 WBm	260

P = 600 CVPD

LDF = 0.5

$$VDF = \frac{2.5 \times 200 + 3.5 \times 400}{600} = 3.167$$

$r = 0.05$

$x = 1.5$

$n = 15$ yrs

$A = P(1+r)^x$

$A = 645.55$

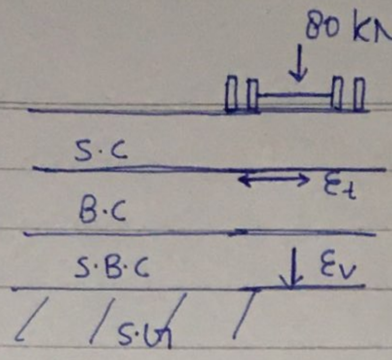
$$CSA = \frac{365A\left((1+r)^n - 1\right)}{r} \times LDF \times VDF \times 10^{-6}$$

$CSA = 8.05 \simeq 8$ msA

③ **IRC METHOD :-** IRC considers two types of failure in Flexible pavements –

1) fatigue Failure in Bituminous layer
2) Rutting due to sub-Grade deformation

To control control Fatigue failure we limit the tensile stress @ Bottom of Bituminous layer and to control Rutting we limit Axial compressive strain At the top of sub-Grade



$$N_F = 2.21 \times 10^{-4} \left(\frac{1}{\varepsilon_t}\right)^{3.89} \left(\frac{1}{M_R}\right)^{0.854}$$

$$N_R = 4.1656 \times 10^{-8} \left(\frac{1}{\varepsilon_v}\right)^{4.5337}$$

} 80% reliability

ε_v → compressive strain @ top of sub-grade
N_F → Fatigue life in no. of standard axles
ε_t → Tensile strain @ Bottom of Bituminous layer
M_R → Resilient Modulus
N_R → Rutting life in no. of standard Axles

Failure criteria as per IRC Method is as follows:-

IRC 37:2001

Fatigue cracking should not occur on more the 20% of pavement Area and Rutting deformation depth should not be > 20 mm in the design life of the pavement

IRC 37: 2012

Fatigue cracking in 20% Area has been considered for traffic upto 30 msA and in 10% Area for traffic beyond 30 msA
For Rutting deformation ≯ 20 mm in 20% of length for traffic upto 30 msA and in 10% of length for traffic beyond 30 MSA

④ California Resistance Value Method

This Method is Based on
1) Stabilometer R-value
2) Cohesiometer C-value

Greater is the value of R & C, Better is the quality of soil

Thickness of pavement with soil having stabilometer and cohisiometer value as R & C is given as

$$t = \frac{K(TI)(90-R)}{C^{1/5}}$$

t → thickness of pavement (cm)
K → const. having value 0.166
TI → Traffic Index = $1.35 (EWL)^{0.11}$
Where EWL → Equivelent wheel load.

$$EWL = \Sigma (AADT \times (EWL)_{const})$$

NO. of Axles	(EWL) const.
2	330
3	1070
4	2460
5	4620

Que. Find EWL for Data given below

NO. of Axle	AADT
2	3750
3	470
4	320
5	120

$EWL = 3750 \times 330 + 470 \times 1070 + 320 \times 2460 + 120 \times 4620$

$EWL = 3.08 \times 10^6$

NOTE for calculation of TI ~~the~~ we work in terms of avg. EWL (i.e $\frac{EWL_{first\ year} + EWL_{last\ year}}{2}$)

$$\left(\frac{t_1}{t_2}\right) = \left(\frac{C_2}{C_1}\right)^{1/5}$$

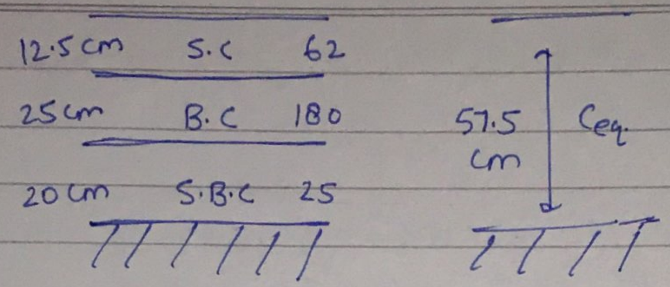
In this Method the equivalency used is

It means t_1 thickness of material C_1 is equivelent to t_2 thickness of material C_2

While Designing we first calculate the total thickness of pavement corresponding to one material (say sub-Base) and then making suitable assumptions and using above equivelency we convert this thikness into thickness of materials to be used in pavement.

Que. calculate equivelent C value of a single layer pavement having same thickness as overall thickness of a 3-layer pavement as shown below.

Material	Thickness	C-value
Bituminous layer	12.5 cm	62
cement treated base	25 cm	180
Well graded gravel	20 cm	25



Bituminous layer

$$h_1 = 12.5\left(\frac{62}{C_{eq}}\right)^{\frac{1}{5}}$$

$$\frac{12.5}{h_1} = \left(\frac{C_{eq}}{62}\right)^{1/5}$$

cement-treated Base layer

$$h_2 = 25\left(\frac{180}{C_{eq}}\right)^{1/5}$$

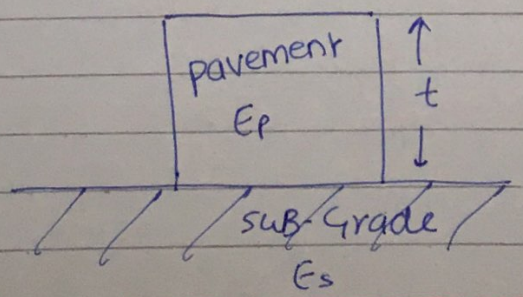
Well-graded layer

$$h_3 = 20\left(\frac{25}{C_{eq}}\right)^{1/5}$$

$$h_1 + h_2 + h_3 = 57.5$$

$$\Rightarrow \quad C_{eq} = 77.457$$

⑤ **Tri-Axial Method** :- As per this Method thickness of pavement with Material having Modulus of Elasticity E_P over a soil-subgrade with Modulus of Elasticity E_s is given as



$$t = \sqrt{\left(\frac{3pxy}{2\pi E_s \Delta}\right)^2 - a^2}\left\{\frac{E_s}{E_P}\right\}^{1/3}$$

t → thickness of pavement in cm
P → Wheel load in kg.
X → Traffic cofficient
Y → saturation cofficient
E_s → Modulus of Elasticity of sub-grade (kg/cm²)
Δ → Design Deflection (cm) (0·25 – 0·5 cm).
Assume Δ = 0·25 cm if not given.
a → Radius of contact Area of Wheel (cm)
E_p → Modulus of Elasticity of pavement

In this Method the equivelency used is as follows:–

$$\frac{t_1}{t_2} = \left(\frac{E_2}{E_1}\right)^{1/3}$$

t_1 → thickness of material with modulus of Elasticity E_1 is equivelent to t_2 thickness of material with Modulus of Elasticity E_2

Que· Design a pavement by triAxial Method using following Data· Wheel load = 4050 kg.
Traffic cofficient = 1·6
saturation cofficient = 0·7
Radius of contact Area of Wheel = 15 cm
Design Deflection = 2·5 mm
Modulus of Elasticity of subgrade = 120 kg/cm²
Modulus of Elasticity of Base course = 360 kg/cm²
Bituminous layer of 7 cm is to be provided @ TOP
Having Modulus of Elasticity = 1200 kg/cm²

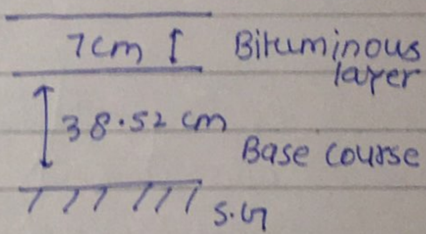
$$t = \sqrt{\left(\frac{3 \times 4050 \times 1.6 \times 0.7}{2 \times 3.14 \times 120 \times 0.25}\right)^2 - (15)^2} \left(\frac{120}{360}\right)^{1/3}$$

$t = 48.96$ cm

let 7 cm of bituminous layer equivelent to h cm of Base course

$$\frac{h}{7} = \left(\frac{1200}{360}\right)^{1/3} \Rightarrow h = 10.46\text{ cm}$$

Hence the pavement will have 7 cm of Bituminous layer and 38.5 cm of Base course



⑥ BURMISTER METHOD:- This Method is Based on elastic two layer system analysis (sub-grade layer and pavement layer)

Wheel load over the pavement is considered as a Flexible plate (Analogy) and formula for Flexible plate in plate Bearing test is used to represent the relation b/w contact pressure and pavement deflection

$$\Delta = \frac{1.5\, p a}{E_s} f$$

$\Delta \rightarrow$ Design deflection (cm)
Assume 0.25 cm if not given.

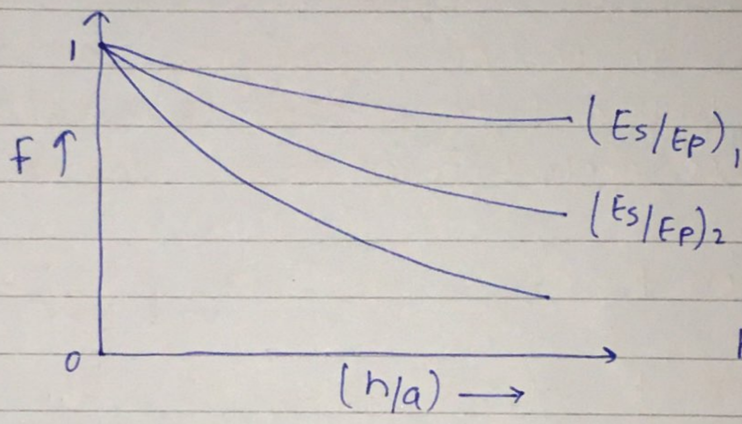
p → contact pressure $\left(\frac{\text{Wheel load}}{\text{contact Area}}\right)$ kg/cm²

15 → for Flexible.

a → Radius of contact Area (cm)

E_s → Modulus of Elasticity of subgrade (kg/cm²)

f → Deflection factor or Displacement factor and it's value depends on Ratios $\left(\frac{h}{a}\right)$ and $\left(\frac{E_s}{E_p}\right)$



h → thickness of pavement (cm)

Calculation of E_s & E_p:-

For this testing is done on the subgrade and pavement layer of an existing pavement having same Modulus of Elasticity. in this we use Rigid plate formula in plate Bearing test to represent the relation b/w pressure used in the test and deflection recorded in the test.

$$\Delta = \frac{1.18\,pa}{E_s} \times f$$

Δ → Deflection recorded in plate bearing test (cm)

p → pressure used in PBT kg/cm²

a → ~~Rigi~~ Radius of steel plates used in PBT (cm)

E_s → Modulus of Elasticity of sub-grade

NOTE When testing is done on sub-grade $f = 1$

Que: PBT conducted with 30 cm Dia plate on a soil subgrade yielded a pressure of 1 kg/cm^2 for 5 mm deflection. Another test was carried over 18 cm thickness of Base-course which yielded a pressure of 5 kg/cm^2 at 5 mm deflection. Design section of a pavement for which wheel load is 4100 kg with a tyre pressure of 6 kg/cm^2 and allowable deflection of 5mm using Burmister's Method.

for $f = 0.2$ & $h/a = 1.2$, $\left(\frac{E_s}{E_p}\right) = 1/80$

for $f = 0.133$ & $E_a/E_p = 80$, $h/a = 2.7$

For testing on subgrade:-

$$\Delta = \frac{1.18\,pa}{E_s} f \qquad 0.5 = \frac{1.18 \times 1 \times 15}{E_s} \times 1$$

$$E_s = 35.4$$

$$\Delta = \frac{1.18\,pa}{E_s} f$$

$$0.5 = \frac{1.18 \times 5 \times 15}{35.4} \times f$$

$$f = 0.2$$

for Existing pavement :-

$$\left(\frac{h}{a}\right) = \left(\frac{18}{15}\right) = 1.2$$

$$E_s/E_p = 1/80$$

For pavement to be Designed

$$\Delta = \frac{1.5 Pa}{E_s} \times f$$

$$0.5 = \frac{1.5 \times 6 \times 14.75}{35.4} \times f$$

$$f = 0.133$$

For $f = 0.133$, $E_s/E_p = 1/80$

$$h/a = 2.7$$

$$h = 39.825 \simeq 40 \text{ cm}.$$

Assume R.F = 1

contact pr. = tyre pr.

= 6 kg/cm^2

$$\text{contact pr.} = \frac{\text{wheel load}}{\pi a^2}$$

$$6 = \frac{4100}{\pi a^2}$$

$$a = 14.75 \text{ cm}$$

Single layer theory :-

In single layer theory Both pavement and sub-grade are assumed to be a single layer with same Modulus of Elasticity. It is also assumed that the pavement layer of thickness z placed over sub-grade is incompressible and the total deflection Δ is only due to the compression of the sub-grade below i.e B/w depth z and ∞ depth

pavement thickness Z corresponding to a given deflection Δ is given as

$$Z = \sqrt{\left(\frac{3P}{2\pi E \Delta}\right)^2 - a^2}$$

NOTE According to Mecleod Method $P_1 n_1 = P_2 n_2$

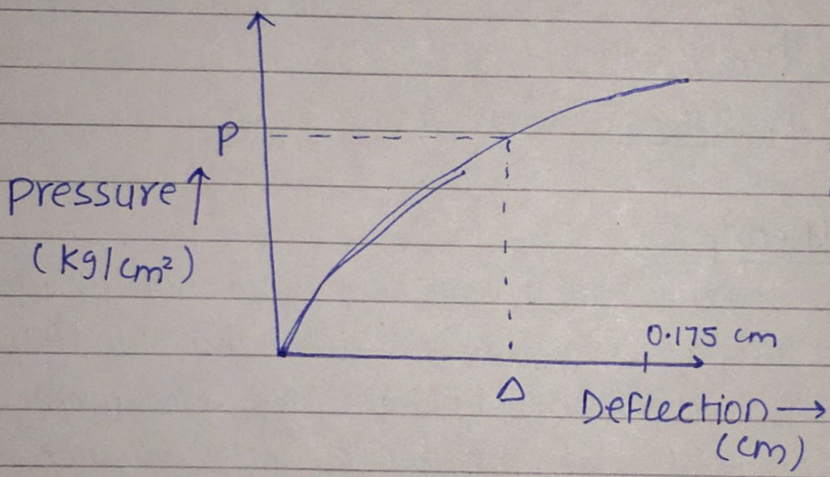
Where P_1 is wheel load which causes failure of pavement with n_1 no. of repeatations and P_2 is wheel load which causes failure of pavement with n_2 no. of repeatations.

Design of Rigid pavements :- IRC :58

(plain jointed concrete pavement)

Rigid pavement is constructed using cement concrete slab. The cement concrete pavement is rested on soil foundation which can be thought of as a viscous or spring foundation (Winkler foundation) having a spring constant K (kg/cm^3)

This k is known as Modulus of sub-Grade reaction. Modulus of sub-Grade reaction is calculated from plate Bearing test corresponding to a deflection of 0.125 cm



$$K = \left(\frac{P}{\Delta}\right)\Big|_{\Delta = 0.125\,cm}$$

The value of K depends on size of plate used in PBT and for Rigid pavements we generally use 75 cm Dia. plate

$$\Delta = \frac{1.18\,Pa}{E_s}$$

$$\left(\frac{P}{\Delta}\right)a = E_s/1.18$$

$$Ka = \text{const.}$$

$$K_1 a_1 = K_2 a_2$$

$$K_{75\,cm} \times \left(\frac{75}{2}\right) = K_{30\,cm}\left(\frac{30}{2}\right)$$

$$\boxed{K_{75} = 0.4\,K_{30}}$$ Gate

IS code recommended

ESE $\boxed{K_{75} = 0.5\,K_{30}}$

Radius of Relative stiffness (l)

Relative stiffness of slab with respect to subgrade is represented by Radius of Relative stiffness

$$l = \left[\frac{Eh^3}{12k(1-\mu)^2}\right]^{1/4}$$

* (M30) / M40

E → Modulus of Elasticity of cement-concrete $= 3 \times 10^5$ kg/cm²

h → thickness of slab (cm)

k → Modulus of sub-grade reaⁿ in kg/cm³

μ → poisson's Ratio = 0.15

Equivelent Radius of Resisting secⁿ (b) :-

Only a small Area of the pavement is eff. in resisting the load through flexure. As per Westergaard equivelent Radius of Resisting secⁿ is given as -

$$b = \sqrt{1.6a^2 + h^2} - 0.675h \quad ; a < 1.724h$$
$$= a \quad ; o/w$$

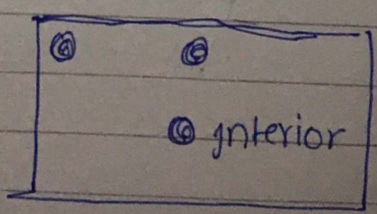
a → Radius of contact Area of wheel

h → thickness of cement concrete slab

WESTERGaurd stresses due to loading :-

3 load positions are considered :-

1) Interior loading
2) Edge loading
3) corner loading



$$\sigma_{int.} = \frac{0.316P}{h^2}\left(4\log_{10}(l/b) + 1.069\right)$$

$$\sigma_{edge} = \frac{0.572P}{h^2}\left(4\log_{10}(l/b) + 0.359\right)$$

} Tensile @ Bottom

$$\sigma_{corn.} = \frac{3P}{h^2}\left(1 - \left(\frac{a\sqrt{2}}{l}\right)^{0.6}\right)$$ → Tensile @ TOP

σ → stress in kg/cm²
P → Wheel load in kg.
l → Radius of Relative stiffness (cm)
b → equivelent Radius of resisting secⁿ (cm)
h → thickness of cement-concrete slab in cm.

TEMPRETURE stresses:- These are developed in cement-concrete pavment due to variation in slab temp. This is caused by –

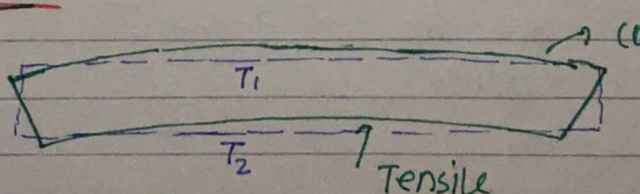
1) Daily variation in Temp.:- It causes Warping (curling)

stress. Due to temp. Gradient accross slab thickness

2) Seasonal variation:- It causes frictional stress due to overall change in slab temp.

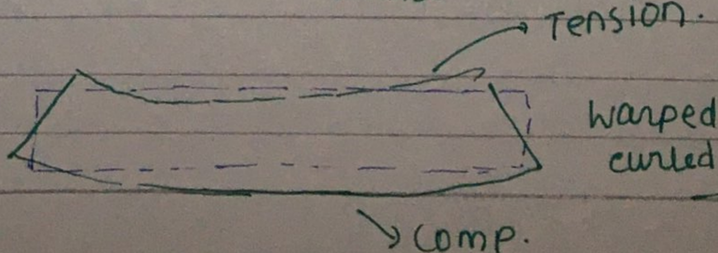
① **Warping stress:-** (Due to Daily variation in Temp.)

Mid-day ($T_1 > T_2$)



warped or curled down

Mid-Night $T_1 < T_2$



warped or curled up.

At Mid Day Top surface will try to expand and Bottom surface will try to contract. which will be restricted by the Weight of the slab, Hence Generating compressive stress @ TOP and Tensile stress @ Bottom.

Similarly During Mid-Night Tensile stress will develop @ TOP and comp. stress @ Bottom

Warping stress is significant near interior & edge location and very less at corner location

- Warping stress in summer is more than working stress in winter due to higher temp. Gradients in summer season.

EQ^n OF Warping stress:-

$$\sigma_{int.} = \frac{E\alpha \Delta t}{2}\left(\frac{C_x + \mu C_y}{1-\mu^2}\right)$$

$$\sigma_{Edge} = \max.\left(\frac{E\alpha \Delta t}{2} C_x , \frac{E\alpha \Delta t}{2} C_y\right)$$

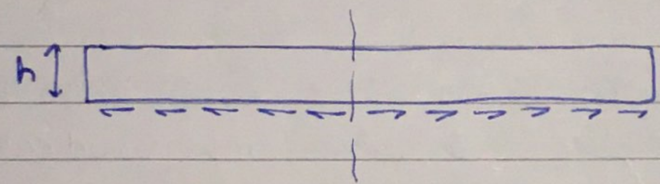
$$\sigma_{corn} = \frac{E\alpha \Delta t}{3(1-\mu)}\sqrt{\frac{a}{l}}$$

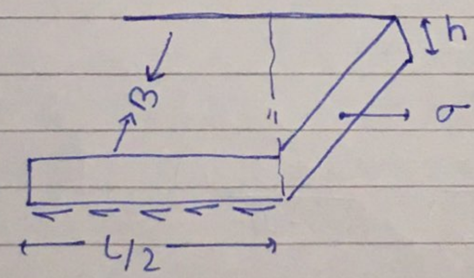
σ → warping stress
E → modulus OF Elastisity
α → coffi of thermal expansion
Δt → temp. Gradient
μ → poisson's Ratio ⇒ 0.15
C_x & C_y → Bread Burry cofficients, depends on $\left(\frac{l_x}{l} \text{ \& } \frac{l_y}{l}\right)$
where l → Radius of Relative stiffness
l_x → Distance b/w transverse joints
l_y → Distance b/w longitudnal joints.

② Frictional stresses :- (due to seasonal variation).

In this case temp. accross the slab thickness is not considered. In summer season the overall temp of the slab will be higher and it will have a tendency to expand. Which will be restricted by Friction generating compressive stresses.

similarly During Winter the overall temp. of the slab will fall and it will have a tendency to contract which will be restricted by friction thereby generating tensile stresses.





$$\sigma \times (Bh) = f\left(\frac{\gamma_c BhL}{2}\right)$$

$$\boxed{\sigma = \frac{f\gamma_c L}{2}}$$

$\sigma \rightarrow$ Frictional stress (comp. in summer, Tensile in Winter)

$f \rightarrow$ friction cofficient b/w slab & soil (1.5)

$\gamma_c \rightarrow$ Wt. density of cement - concrete slab (24 kN/m³)

$L \rightarrow$ Distance b/w transverse joints.

Frictional stress is significant near central portion of the slab and Negligible Near corner Locations

Critical combination of stresses :-

Out of various Wheel load stresses corner stress is Max^m, interior stress is Min. and edge stress is in intermediate Range.
In case of temp. stress interior stress is max. corner stress is Min. and edge stress is in intermediate Rang.

Hence in combination of Wheel load and temp. stress Edge Region comes out to be most critical.
Therefore for Design we consider Edge & corner location (corner because of Discontinuity)

Various critical combination of stresses are:-

1) Summer Mid-Day Edge region

$$\sigma_{Bottom} = \sigma_{load} + \sigma_{warping} - \sigma_{Frictional}$$

2) Winter Mid-Day Edge Region

$$\sigma_{Bottom} = \sigma_{load} + \sigma_{warping} + \sigma_{Frictional}$$

3) Summer Mid Night corner region

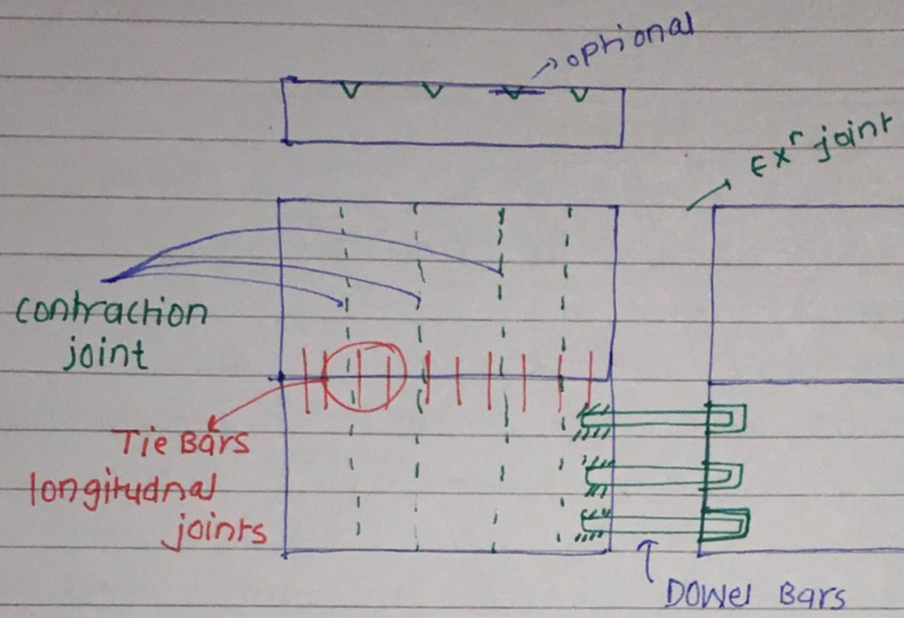
$$\sigma_{Top} = \sigma_{load} + \sigma_{warping}$$

Out of the above 3 combination generally the 1st combination comes out to be most critical as summer Warping stress of Edge is more than winter warping stress of Edge.

$+ \rightarrow$ Tension

$- \rightarrow$ Compression

Various Type of joints in cement-concrete pavement :-



Various types of joints provided in cement concrete pavements are

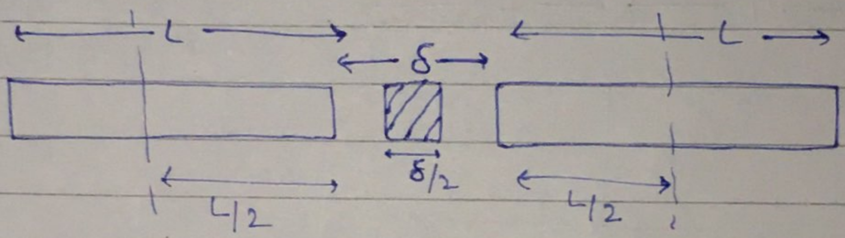
1) Transverse joint :-
   a) contraction joint
   b) Expansion joint
   c) construction joint (Tried to Match with contraction and Expansion joint)
2) longitudnal joint

NOTE If CVPD > 450 then IRC recommends use of dowbel Bars even at contraction joint

① Expansion joint :- purpose is to allow for the expansion of pavement due to rise in temprature with Respect to construction Temprature.
As per IRC Max. spacing b/w Expansion joint is 140 m and Max. gap of joint is 2.5 cm

At expansion joints dowel Bars are used Which develops Bending, Bearing and shearing stress and helps in load transfer

One end of the dowel bar is bonded with concrete and the other end is free to move. Fillers are provided to seal the expansion joint. These fillers are assumed to be compressed by 50% of their thickness hence the Gap of joint should be twice the expansion is concrete.



$$\frac{\delta}{2} = L\alpha(\Delta t)$$

$$\boxed{\delta = 2L\alpha(\Delta t)}$$

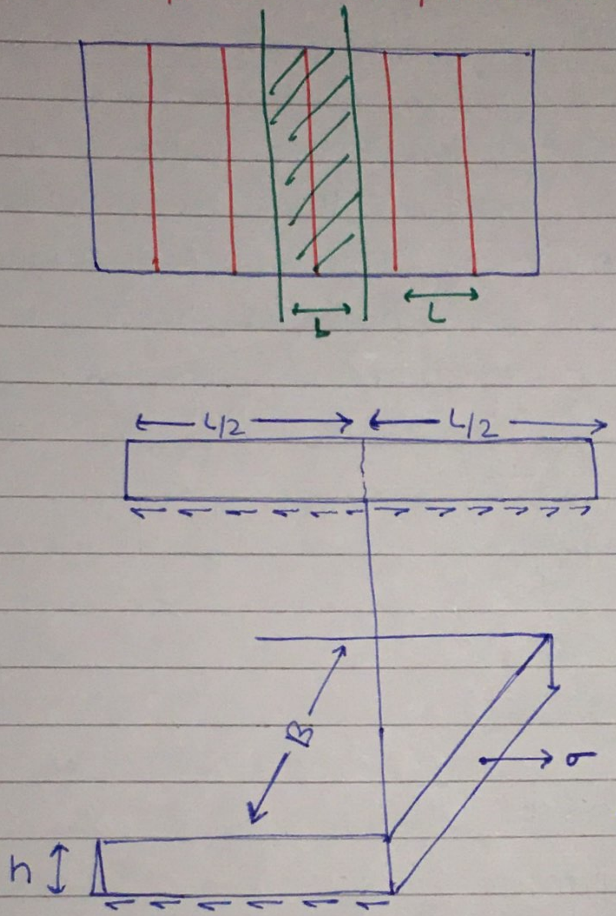
$L \rightarrow$ spacing b/w expansion joints
$\Delta t \rightarrow$ Temp. change
$\delta \rightarrow$ Gap of expansion joint

② **Contraction joint** :- It is provided to control cracks due to shrinkage. To regulate the crack that is to ensure that crack develops @ predetermined location, the slab is weakened @ certain locations these locations are called contraction joints.

NOTE During initial curing period shrinkage occurs in concrete and if this shrinkage is restricted tensile stress is Generated. Fall in temp. also generates Tensile stress.

Spacing b/w contraction joint:-

A) When R/F is not provided.



L → spacing b/w contraction joint

σ_c → permissible tensile stress in cement concrete

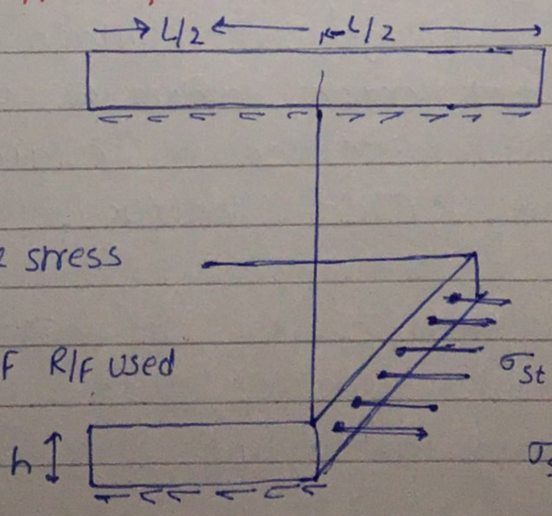
γ_c → Wt density of cement concrete ($0.8\ kg/cm^2$)

f → friction @ interface

$$\sigma B h = \frac{f \gamma_c B h L}{2}$$

$$L = \frac{2\sigma_c}{f\gamma_c}$$

B) When R/F is provided



h → thickness of slab

B → width of slab.

σ_{st} → per. tensile stress in steel

A_{st} → C/s Area of R/F used

$\sigma_{st} \cdot A_{st}$

$$\sigma_{st} \cdot A_{st} = \frac{f \gamma_c B h L}{2}$$

$$L = \frac{2\sigma_{st} A_{st}}{f \gamma_c B h}$$

NOTE When R/F is not provided the Max. spacing b/w contraction joints as per IRC can be 4.5 m

Que. The Max. increase in Temp. is expected to be 26°c after the construction of C-C pavement. if the expansion joint Gap is 2.2 cm. Design the spacing b/w expansion & contraction joint. $\alpha = 10\times10^{-6}/°c$

$\gamma_c = 2400\ kg/m^3$, $\sigma_c = 0.8\ kg/cm^2$, $f = 1.4$

Expn joint

$\delta = 2L\alpha(\Delta t)$

$2.2\times10^{-2} = 2\times L\times(10\times10^{-6})\times 26$

$L = 42.3\ m$

Adopt $L = 42\ m$.

Contraction

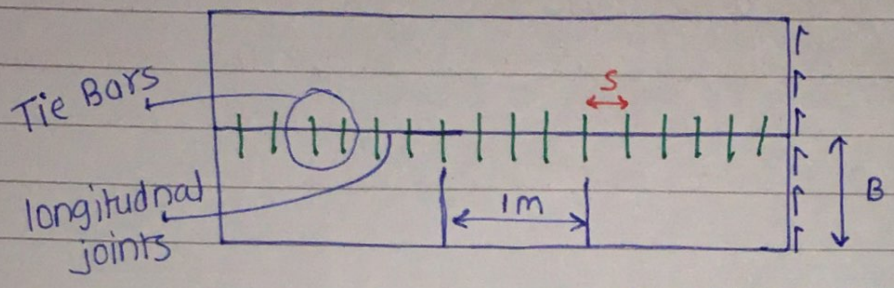
$$L = \frac{2\sigma_c}{f\gamma_c} = \frac{2\times0.8\times10^4}{1.4\times2400} = 4.7\ cm$$

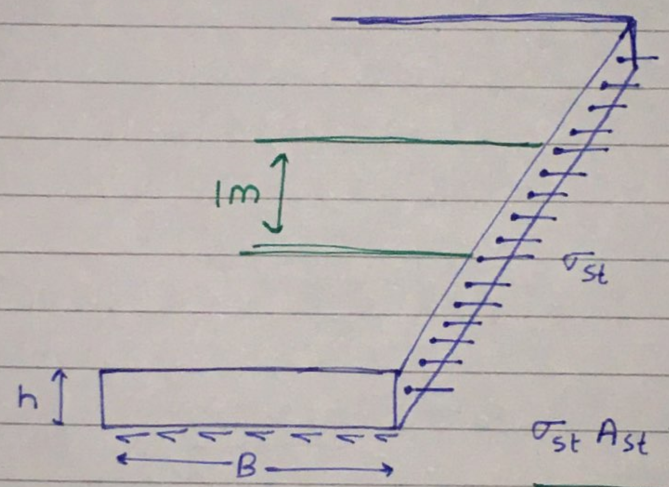
Adopt $L = 4.5\ m$ $\ngtr 4.5\ cm$ (IRC)

③ **longitudnal Joints:-** These are provided along the length of pavement. They reduce Warping stress (Hence also called Warping joints) Functions as contraction joint and lane demarkation Normall Width of slab is kept b/w 3-5 m Hence for Width Greater than that longitudnal joint become necessary. Tie Bars are provided at longitudnal joints to ensure that slab remains together firmly

- Tie Bars are not designed as load transfer devices load is transferred through agreegate interlocking to the adjacent slab

Tie Bars are bonded with concrete and we generally use deformed bars of Dia (10-16 mm). They are designed to withstand tensile stress due to friction b/w slab & soil





$$\sigma_{st} A_{st} = f \gamma_c (Bh \times 1)$$

A_{st} → c/s Area of Tie Bars used per m length of slab

$$A_{st} = \frac{f \gamma_c B h}{\sigma_{st}}$$

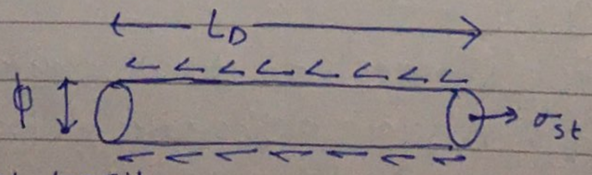
σ_{st} → per. Tensile stress in steel

Spacing B/w Tie Bars :-

$$S = \frac{1000}{\left(\frac{A_{st}}{\frac{\pi}{4}\phi^2}\right)}$$ (mm)

ϕ → Dia of Tie Bar

length of Tie Bars :-



$$\sigma_{st} \times \frac{\pi}{4}\phi^2 = \tau_{bd}(\pi \phi \times L_D)$$

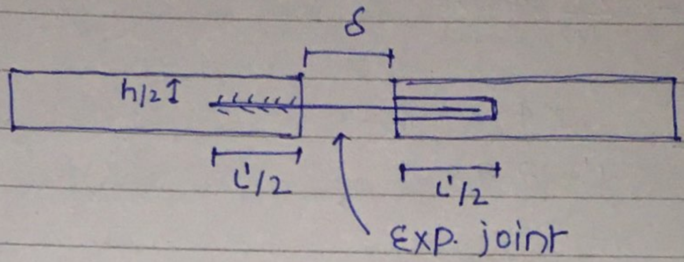
L_d → development length

τ_{bd} → Bond stress

$$L_D = \frac{\sigma_{st}\phi}{4\tau_{bd}}$$

length of Tie Bar = $2L_D$ (when there is no gap)

Design of Dowel Bar:-



length of dowel bar (L) = L' + δ

L' → Embaded length
δ → Gap of joint
L → Total length of dowel bars

- Dowel bars are designed as load transfer device across the transverse joint
- They keep the two slab @ same height & help the load transfer
- Dowel Bars are mild steel rounded Bars Bounded on one side and free on other.
- Normally 25-40 mm dia and 400-500 mm length is used
- The stresses in dowel Bars are
  - a) Shearing
  - b) Bending
  - c) Bearing

BradBurry Analysis:- load transferred by a single Dowel bar (100% effective) is given as —

$$P_s = \frac{\pi}{4} \phi^2 \sigma_s$$

$$P_b = \frac{2\phi^3}{L' + 8.8\delta} \sigma_b$$

$$P_{br} = \frac{L'^2 \phi}{12.5 (L' + 1.5\delta)} \sigma_{br}$$

P_s, P_b, P_{br} → are load transferred in shearing, Bending, Bearing respectively.

$\sigma_s, \sigma_b, \sigma_{br}$ → prmissible stresses in shearing, Bending & Bearing

L' → embaded length

ϕ → Dia of Dowel Bar

δ → Gap of joints

DEsign steps:-

(i) Embaded length of dowel bar is decided by equating the load transferred in bending and bearing and making suitable assumptions.

$$P_b = P_{br}$$

$$\frac{2\phi^3}{L' + 0.88\delta} \sigma_b = \frac{L'^2 \phi}{12.5 (L' + 1.5\delta)} \sigma_{br}$$

$$L' = 5\phi \sqrt{\frac{L' + 1.5\delta}{L' + 0.88\delta} \frac{\sigma_b}{\sigma_{br}}}$$

2) Making suitable assumptions about ϕ & δ
($\phi = 25$ mm, $\delta = 20$ mm)
l' can be calculated by trial & Error
length of dowel bar = $l' + \delta$ *

3) calculate P_s, P_b and P_{br}
4) load transfer capacity of a Dowel Bar system is assumed to be 40% of wheel load (P)
Hence no. of 100% effective Dowel bars required

$$= \text{Max}\left(\frac{0.4P}{P_s}, \frac{0.4P}{P_b}, \frac{0.4P}{P_{br}}\right)$$

This value is called load capacity factor

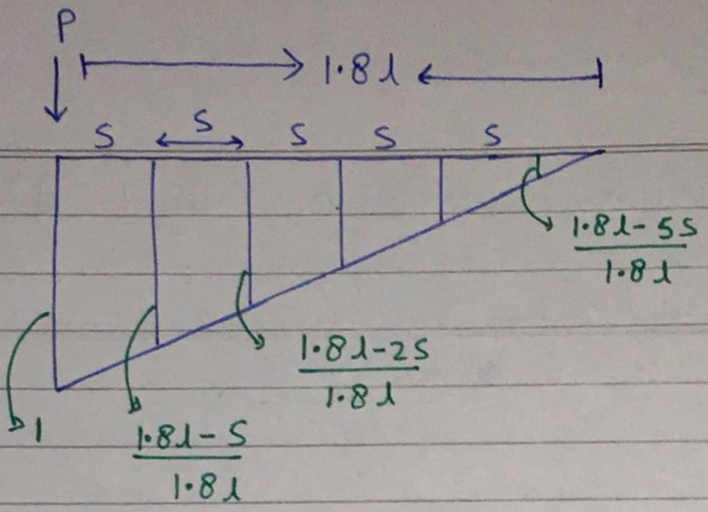
5) Distance on either sides of wheel load upto which Group of dowel Bars are effective in load transfer is theortically taken as $1.8\,l$, where $l \rightarrow$ Radius of Relative stiffness

In exam adopt $1l$ if Not given

NOTE However IRC recommends $1l$ for design calculations because there will be some loosning due to expansion and contraction of pavement.

Load capacity of a dowel bar is taken as 1 (i.e 100% effective) just below the wheel load and is taken as zero @ a distance of $1.8\,l$ from the wheel load.

In b/w 100% effective and 0% effective Dowel Bars load capacity is assumed to vary linearly.
For Design edge loading is taken as critical position.
Hence. Group of Dowel Bars in length $1.8\,l$ should be able to transfer 40% of wheel load.



$$\text{Actual load capacity} = 1 + \frac{1.8l - s}{1.8l} + \frac{1.8l - 2s}{1.8l} + \cdots + \frac{1.8l - 5s}{1.8l}$$

spacing b/w Dowel bars is so (S) selected that the actual load capacity > load capacity Factor

Que: Design a Dowel Bar system for the following cond.
Design wheel load = 51 KN, Design load transfer = 40%.
slab thickness = 28 cm, Gap of joint (δ) = 2 cm
permissible Flexural stress in dowel Bar = 140 MN/m²
permissible shear stress in dowel Bar = 100 MN/m²
permissible Bearing stress of concrete = 10 MN/m²
effect of load goes upto 1.8 l, Modulus of subgrade reacⁿ = 80 MN/m³, Modulus of Elasticity = 30000 MN/m²
poisson's Ratio = 0.15

$$l = \left[\frac{Eh^3}{12K(1-\mu^2)}\right]^{1/4}$$

$$l = \left[\frac{30000 \times 0.28^3}{12 \times 80 \times (1-0.15)^2}\right]^{1/4} = 0.915$$

$\phi = 25\,mm$, $\delta = 20\,mm$

$$L' = 5\phi \sqrt{\frac{L' + 1.5\delta}{L' + 8.8\delta} \frac{\sigma_b}{\sigma_{br}}}$$

$L' = 0.405$

length of dowel bar = $L' + \delta = 0.425$ m

$L = 425$ mm

$$P_s = \frac{\pi}{4}\phi^2 \times \sigma_s = \frac{\pi}{4} \times 0.025^2 \times 100 \times 10^3 \text{ kN/m}^2$$

$P_s = 49$ kN

$$P_b = \frac{2\phi^3}{L' + 8.8\delta}\sigma_b = \frac{2 \times 0.025^3}{0.405 + 8.8 \times 0.02} \times 140 \times 10^3 \text{ kN/m}^2$$

$P_b = 7.53$ kN

$$P_{br} = \frac{L'^2 \phi}{12.5(L' + 1.5\delta)} \times \sigma_{br} = \frac{0.405^2 \times 0.025}{12.5(0.405 + 1.5 \times 0.02)} \times 10 \times 10^3 \text{ kN/m}$$

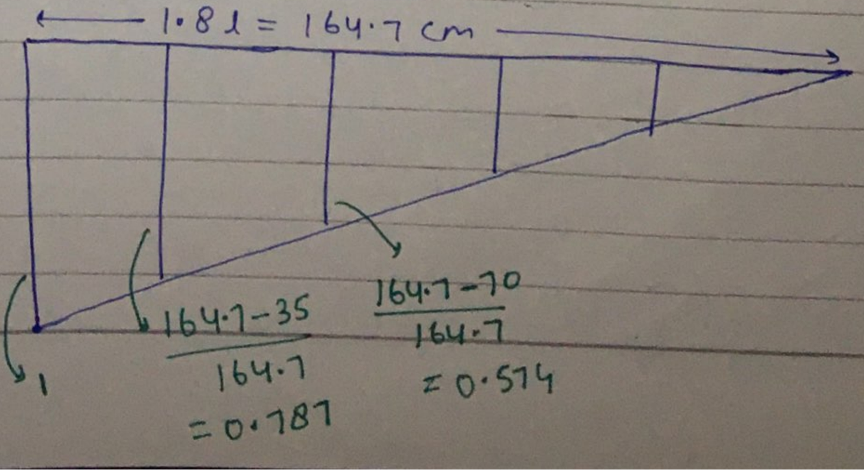
$P_{br} = 7.54$ kN

$$\text{load capacity factor} = \text{Max.}\left\{\frac{0.4P}{P_s}, \frac{0.4P}{P_b}, \frac{0.4P}{P_{br}}\right\}$$

$= 2.71$

Assume $S = 35$ cm

$1.8l = 164.7$ cm



Actual load capacity $= 1 + \frac{164.7 - 35}{164.7} + \frac{164.7 - 35 \times 2}{164.7} + \cdots$

$$= 1 + \frac{4 \times 164.7 - 35 \times 10}{164.7}$$

$= 2.87 >$ load capacity factor

Hence, Accepted!

MATERIALS

Desirable properties of Aggregates :-

I) **Strength** :- Aggregate should posses high resistance to crushing to withstand stress due to wheel loads

II) **Hardness** :- Aggregate should be Hard enough to resist abrasive action by traffic movement

III) **Toughness** :- Resistance of Aggregates to impact is called toughness. Aggregates should be able to resist effects caused by jumping of steel tyred wheels

iv) **Shape of Aggregates** :- Too flaky and too elongated particles should be avoided as we've less strength and durability.

v) **Adhesion with Bitumin** :- Agg. should ~~be~~ have less affinity for water when compared to bitumin to present stripping.

vi) **Durability** :- property of aggregate to withstand adverse effect of weathering is called soundness, Agg. should be sound enough to withstand weathering actions

vii) **Freedom from deleterious particles**

Aggregates should be free from dust, clay and other objectionable materials.

Various Aggregate Tests are:-

(i) crushing Test :- To find crushing strength.

load is applied on a sample (40 tonne at 4 tonnes/min) and then the sample is sieved through 2.36 mm sieve

$$\text{Agg. Crushing Value} = \frac{\text{Wt. passing } 2.36\text{mm sieve}}{\text{Initial weight}} \times 100$$

IRC Recommendation:-

ACV ≯ 30% for surface course

ACV ≯ 45% for Base course

(ii) Impact Test :- To determine toughness

After applying Impact load (13.5-14 kg Hammer, 38 cm freefall and 15 Blows) the sample is sieved through 2.36 mm sieve

$$AIV = \frac{\text{Wt. Passing } 2.36 \text{ mm sieve}}{\text{Initial Weight}} \times 100$$

IRC Recommendations:-

AIV ≯ 30% for Bearing course

AIV ≯ 40% for Bituminous Macadam.

(iii) Abrasion Test :- To determine Hardness

Various abrasion tests are

a) los Angles Abrasion test

b) Deval's Abrasion test

c) Dorry's Abrasion test

$$LAAV = \frac{\text{Wt. passing } 1.7 \text{ mm sieve}}{\text{Initial weight}} \times 100$$

IRC recommendations :-

Max. Value of 35% is allowed for Bituminous concrete and 40% for WBM Base course

(IV) **Soundness Test :-** To determine resistance against Weathering (Durability)

Aggre. are subjected to alternate cycles of wetting in saturated solution of sodium sulphate or Magnisium Sulphate for 16-18hr. and then drying over an oven at 105-110°C to a constant mass.

After 5 cycles loss in wt. of aggregate is determined by sieving the sample

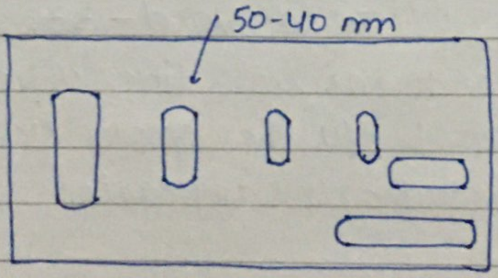
IRC recommendation :-

loss in wt. ≯ 12% for sodium sulphate

≯ 18% for Magnisium sulphate

(V) **Shape test :-**

a) Flekiness index :- It is defined as % by wt of Aggre. particles whose least dimension < 0.6 × it's Mean dimension

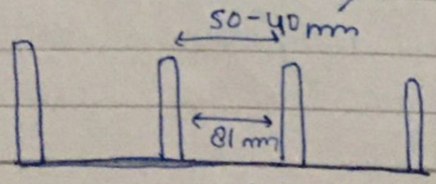


$= 0.6 \times \frac{50+40}{2}$

$= 27$

$$F.I = \frac{\text{Wt. passing}}{\text{Initial Wt.}}$$

b) Elongation Index :- It is defined as % by wt. of particles whose Greatest dimension > 1.8 × it's Mean dimension



$1.8\left(\frac{50+40}{2}\right)$

$= 81$ mm

$$EI = \frac{\text{Wt. retained}}{\text{Initial Wt.}} \; **$$

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Elongation index is calculated on a sample from which Flaky particles have been removed.

* Combined Index = FI + EI

Elongated and flaky particles are less workable and are ~~under~~ likely to break under small loads

FI and EI values in access of 15% is Generally considered undesirable.

MORTH has specified the Max. permissible value of the combined index of course Aggregates as 30% for WMM Base course, DBM Binder course and BC surface course

(vi) Angularity Number :- Angularity is absense of Roundness.

It represents degree of packing.

$$\text{Angularity Number} = 67 - \%\ \text{solid volume}$$

$$= 67 - \frac{100 \times W}{C \times G_a}$$

$$= \%\ \text{void} - 33$$

$W \rightarrow$ wt. of Aggr. to fill a 3 litre cylinder

$C \rightarrow$ wt. of Water to fill the empty cylinder

$G_a \rightarrow$ specific Gravity of aggregate

* NOTE: • Angularity Number varies from 0–11 with 7–10 being preferred.

• For flexible pavement we prefer High Angularity No. For better interlocking and for Rigid pavements we prefer low angularity No. For Better strength & workability

Fuller's Max. Density:- The properties of Bituminous mix is very much dependent on the aggregates and their grain size distribution. A dense mix may be obtained when particle size distribution followes Fuller's curve given by

$$P = \left(\frac{d}{D}\right)^m \times 100$$

Where $P \rightarrow$ % finer than d_{mm} in the material

$D \rightarrow$ Max. size of aggregate (mm)

$m \rightarrow$ parameter depending upon shape of Aggre.

(Assume 0.5 if not given)

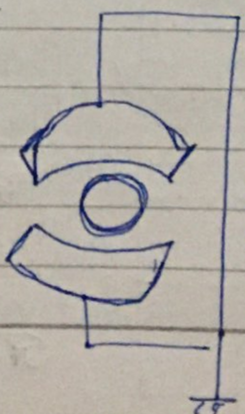
Que. For an aggregate soil Mixture with Max. size of Aggregate as 60 mm. the % of material b/w sizes 6 mm and 75 µ for obtaining Fuller's Max. Density is ?

$$P_{(0.075-6)} = \left\{\left(\frac{6}{60}\right)^{0.5} - \left(\frac{0.075}{60}\right)^{0.5}\right\} \times 100$$

$$= 28\ \%$$

Marshall Mix Design:- It is a Bituminous mix design Method which helps in evaluation of performance of Bituminous Mix and in deciding optimum Bitumin content. For this two tests are performed

1) stability test :-

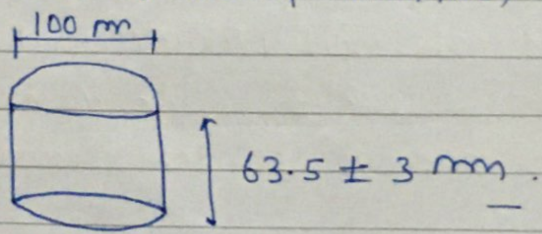


stability is defined as Max. load in kg. carried by the speciman before failure @ 60°C

2) **Flow Test :-** Flow is defined as deformation in units of 0.25 mm b/w no load and Max^m load used in stability test.

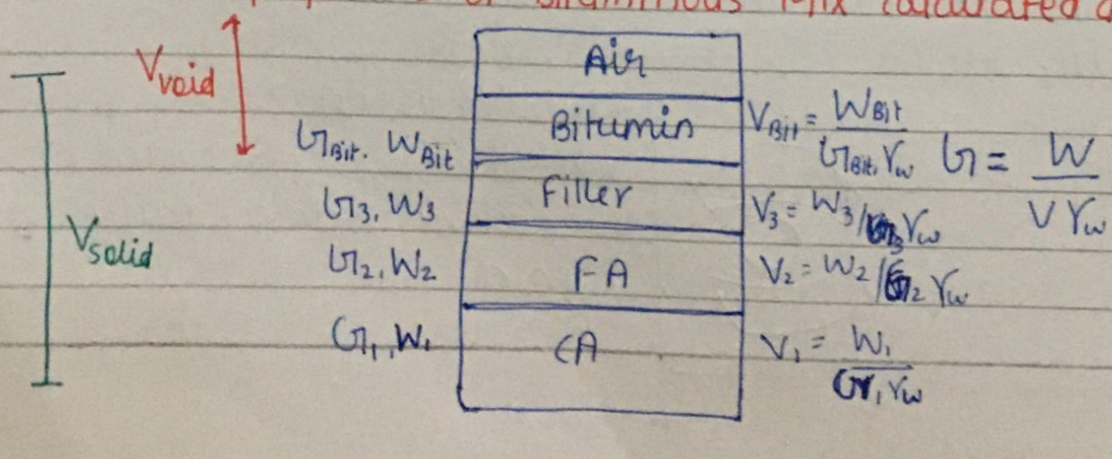
To prepare this speciman 1200 grams of Aggre. (course Aggre + Fine Aggre. + Filler) is heated at 175 – 190 °C. Bitumin is also heated to 121 – 125 °C and mixed with Mineral Aggregates. The initial % of Bitumin is taken b/w 3.5 – 4 % by wt. of Aggregates. The heated aggre. and Bitumin is thorougly mixed at a temp. of 154 – 160 °C

The mixture is placed in preheated mould and compacted by hammer with 50 Blows on either side (75 Blows in case of heavy traffic)



In every successive speciman tested Bitumin content is increased by 0.5 %. Graphs are plotted in terms of Various parameters against the % of Bitumin and then the optimum Bitumin content is obtained of the criteria given by marshall mix design.

Various properties of Bituminous Mix calculated are :-



Theoritical specific Gravity (G_t) :- It is the specific Gravity calculated excluding Air voids

$$G_t = \frac{\text{Wt. of Mix}}{\text{solid Vol}^m \, \gamma_w}$$

$$\boxed{G_t = \frac{W_1 + W_2 + W_3 + W_{Bit}}{\frac{W_1}{G_1} + \frac{W_2}{G_2} + \frac{W_3}{G_3} + \frac{W_{Bit}}{G_{Bit}}}}$$

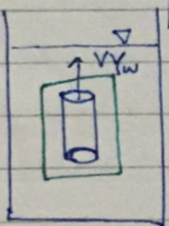
Bulk specific Gravity (G_m) :- It is the specific Gravity calculated on Total volume

$$G_m = \frac{\text{Wt. of Mix}}{\text{Total Vol}^m \times \gamma_w}$$

Wt. of sample in water = Wt. of sample in air − $V\gamma_w$

$V\gamma_w$ = Wt. of sample in Air − Wt. of sample in Water

$$\boxed{G_m = \frac{\text{Wt. of Sample in Air}}{\text{Wt. of sample in Air} - \text{Wt. of sample in Water}}}$$



NOTE $\boxed{G_m \leq G_T}$

percentage Air voids :-

$$V_v = \frac{\text{Vol. of Air}}{\text{Total Vol}^m} \times 100$$

$$V_v = \frac{\text{Total Vol}^m - \text{Vol}^m \text{ of solid}}{\text{Total Vol}^m} \times 100$$

$$V_v = \left(1 - \frac{\text{solid Vol}^m}{\text{Total Vol}^m}\right) \times 100$$

$$\boxed{V_v = \left(1 - \frac{G_m}{G_T}\right) \times 100}$$

percentage Volume of Bitumin :- (V_b)

$$V_b = \frac{Vol^m \text{ of Bitumin}}{\text{Total vol.}^m} \times 100$$

$$V_b = \frac{W_{Bit} / G_{Bit} \gamma_w}{\frac{W_1 + W_2 + W_3 + W_{Bit}}{G_m \gamma_w}}$$

$$\boxed{V_b = \frac{W_{Bit}}{\text{wt. of Mix}} \times \frac{G_m}{G_{Bit}}}$$

Voids in mineral Aggregates :- (VMA)

$$VMA = \frac{Vol.^m \text{ of voids}}{\text{Total vol.}^m} \times 100$$

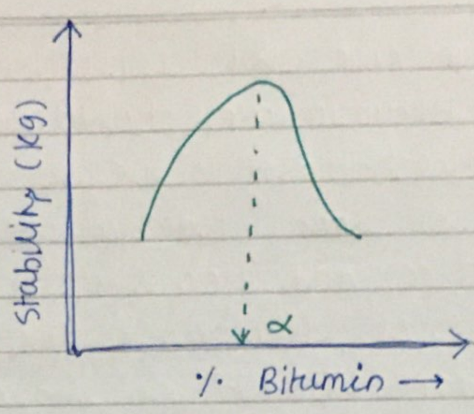
$$VMA = \frac{V_{air} + V_{bit}}{\text{Total vol.}^m} \times 100$$

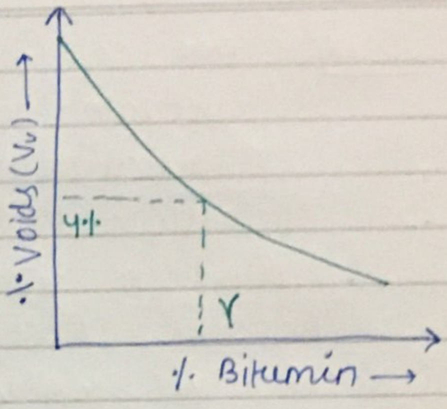
$$\boxed{VMA = V_v + V_b}$$

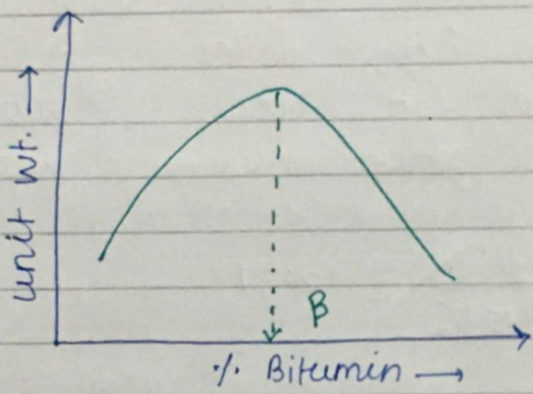
Voids Filled With Bitumin :- (VFB)

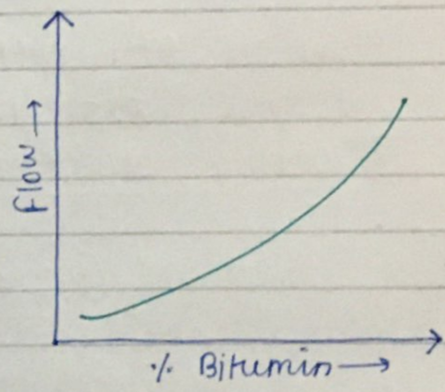
$$VFB = \frac{V_{Bit.}}{Vol^m \text{ of voids}} \times 100$$

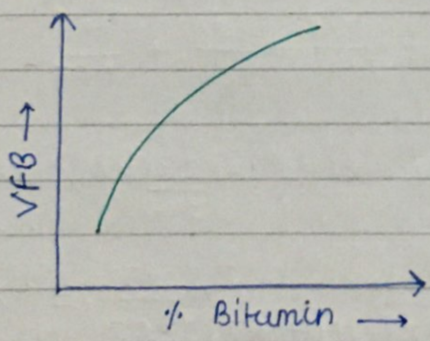
$$\boxed{VFB = \frac{V_b}{VMA} \times 100}$$

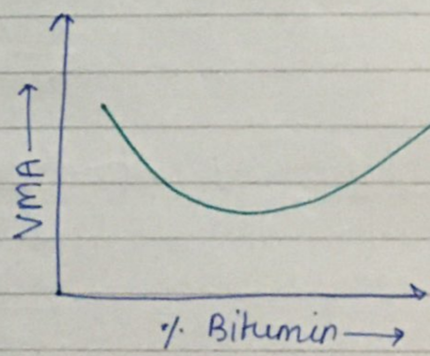












* Optimum Bitumin % $= \frac{\alpha + \beta + \gamma}{3}$

Marshall Mix design specifications:-

Stability → Min. 340 kg

Flow → 8-16 (deformation 2mm → 4mm)

% Air voids → 3-5%

Voids filled with Bitumin → 75-85%

Que. Specific Gravities and wt. proportions for aggregates and Bitumin are shown below for the preparation of Marshall Mix Design.
The vol. and wt. of one Marshall speciman was found to be 475 cc and 1100 grams.
Assuming Absorption of Bitumin in Aggre. to be Zero calculate V_v, V_b, VMA, VFD = ?

Item	CA_1	CA_2	FA	Filler	Bitumin
Wt. (gm)	825	1200	325	150	100
sp. Gravity	2.63	2.51	2.46	2.43	1.05

$$G_m = \frac{1100}{475\,\gamma_w} = 2.31$$

$$G_t = \frac{825+1200+325+150+100}{\frac{825}{2.63}+\frac{1200}{2.51}+\frac{325}{2.46}+\frac{150}{2.43}+\frac{100}{1.05}} = 2.405$$

$$V_v = \left(1-\frac{G_m}{G_t}\right)\times 100 = \left(1-\frac{2.31}{2.405}\right)\times 100 = 3.95\%$$

$$V_b = \frac{100}{825+1200+325+150+100} \times \frac{2.31}{1.05} = 8.46\%$$

$$VMA = V_v + V_b = 12.41\%$$

$$VFB = \frac{V_b}{VMA}\times 100$$

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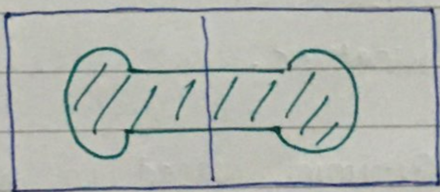
Properties of Bitumin :-

1) **Viscosity** :- It is the property of Bitumin which resists flow due to internal friction. It is estimated using sliding plate viscometer or E-flux viscometer

VG	10	20	30	40
			↑ Road construction	→ Intersection Toll Booths.
Min. Viscosity (poise)	800	1600	2400	3200

VG (30) ⟶ (100 ± 20) 30

ii) **Ductility** :- Bitumin Binder should be sufficiently Ductile i.e It should be capable of being stretched without Breaking.



The distance in cm the Briquette can be stretched at 27°c underwater @ 5cm/Min. without breaking is called Ductility

It varies from 5-100 and a Min. value of 50 is commonly specified.

iii) **penetration** :— It is an indirect Measure of hardness

A penetration Grade 80/100 means that the penetration of 100 gm Needle when left for 5 sec. @ 25°c in Bitumin was 8-10 mm.

safe limit for heating Bitumin is 50° under flash point

iv) Specific Gravity:- Estimated from pycnometer method @ 27°c, it varies from (0.97 → 1.02)

v) loss on Heating:- It should not be more than 1% When 50 grm. bitumin is heated @ 163°c over an oven for 5 hr.

vi) Water content:- To avoid foaming the Max. water content in Bitumin should not exceed 0.2% by weight

Cut-Back Bitumin:- Bitumin in valatile Dilutent like kerosin, Diesel. etc.
(cold weather)
• used in cold weather.

vii) Emulsion:- Aquas Bitumin. Used in wet condition.

Grades of TAR:- It varies from RT1 (lowest viscosity & used in surface paintings) to RT5 (Highest Viscosity used in Grouting)

Bitumin	TAR
• Obtained from fractional distilation of crude oil	Obtained from Distructive distilation of coal or wood
• soluble in carbon di sulphide and carbon tetra chloride	soluble in Toluene
• Insoluble in Water	• Insoluble in water. contains more free carbon content.
• More resistent to temp. & Water	

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Asphalt pavement distresses :-

i) **Bleeding** :- Means Migration of excess Bitumin from the Mix to the Road surface which is deposited as a thin shiny film making the Road slippry.

It is caused by—

i) excesses
ii) low Air void
iii) use of low viscosity Bitumin
iv) Too heavy Tack-coat

ii) **Remedy** :- Apply hot sand & roll it during hot weather to layout extra asphalt binder at the surface

iii) **Ravelling**:— It is progressive disintegration of asphalt surface which is the result of dislodgement of aggregate particle in the mix at the surface. It occurs due to lack of sufficient cohesion within the asphalt mix due to inadequate binder contact low density, lack of fines ang aging, However Ravelling due to aging occurs after many years.

iv) **Stripping** :- It is the breaking of adhesion b/w aggregate and asphalt binder, usually in the presence of Moisture.

v) **Scaling** Thin wearing course seprates due to intrusion of Moisture b/w binder and wearing course.