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_0)

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 $= \dfrac{\mu_{\text{Braking test}}}{\mu_{\text{max. known}}} \times 100$

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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

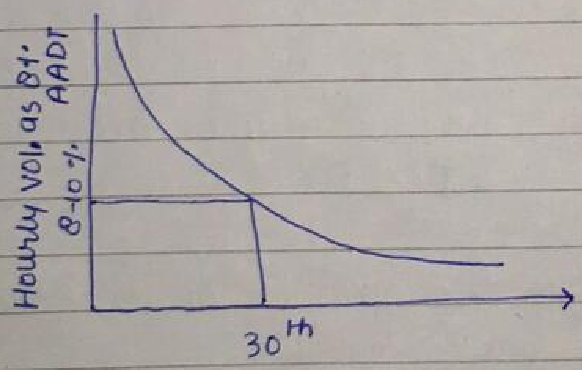
(iii) Trend chart :

Vol. trend over a period of year is calculated

It helps in planning expansions and regulations

(iv) 30^{th} 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

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

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

$$\text{Avg. 24hr vol}^m \text{ of Tuesday} = \frac{400\times29+535\times22.5+650\times18.8+710\times17.1+650\times18.5}{5}$$

$= 12007.3$ 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 diff^n 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

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

n → no. of vehicles crossing a perticular location in a given interval of time

V_i → 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

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

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

Reciprocal of SMS Gives the avg. travel time (per KM)

NOTE ① since Airthemetic Mean $\geq$ Harmonic Mean

*** Hence TMS $\geq$ 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

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

$$V_s = \frac{12}{1\times\frac{1}{3.5} + 4\times\frac{1}{8.5} + 0\times\frac{1}{11.5} + 7\times\frac{1}{15.5}} = 9.434 \text{ km/h}$$

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

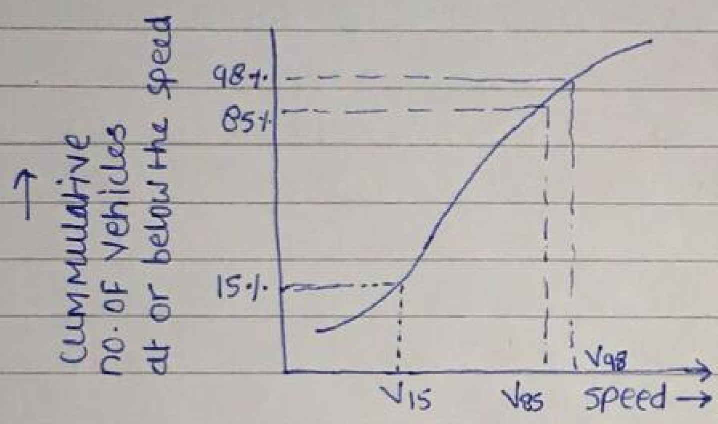
- It includes stop delays
- It is used to analyse traffic Flow condition.
- Running speed $\geq$ Journey speed

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.

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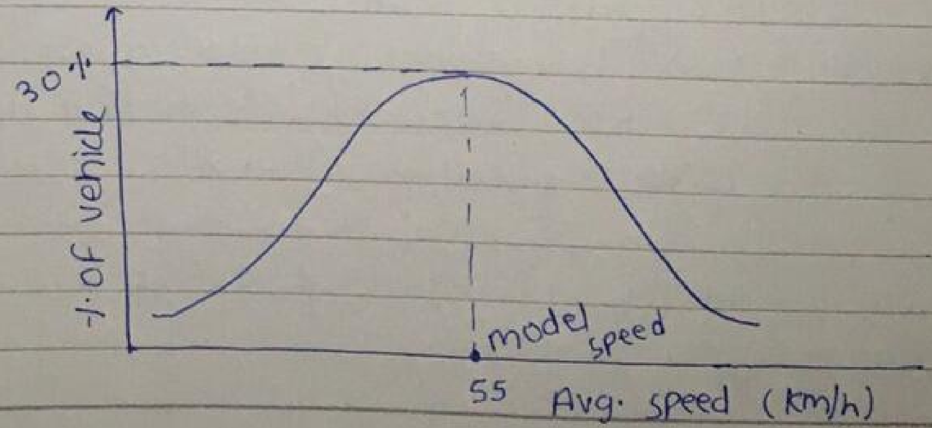
	Speed Range (km/hr)	No. of vehicles	% of vehicles	Cummulative %
5	0-10	12	$\frac{12}{850} \times 100 = 1.41$	1.41
15	10-20	18	2.11	3.52
25	20-30	68	8	11.52
	V_{15}			15%
35	30-40	89	10.48	22
45	40-50	204	24	46
55	50-60	255	30	76
	V_{85}			85%
65	60-70	119	14	90
75	70-80	43	5.06	95.06
	V_{98}			98%
85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

· 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.

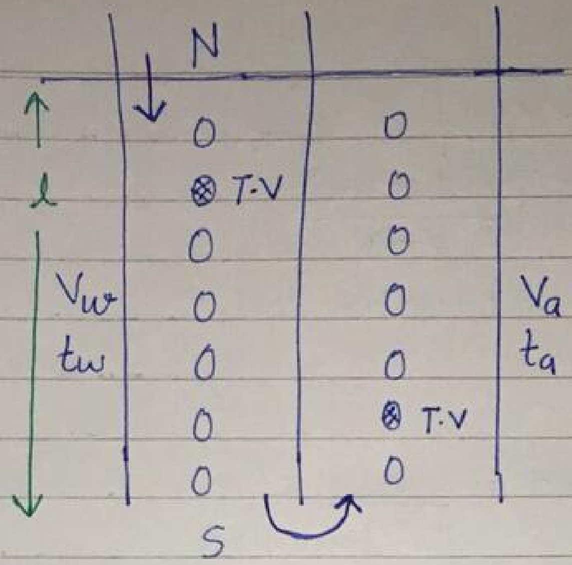
observer1 → Time at control points and amount of delay

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)

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

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

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

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

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

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

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

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

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

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

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

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

From eqⁿ ①

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

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

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

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

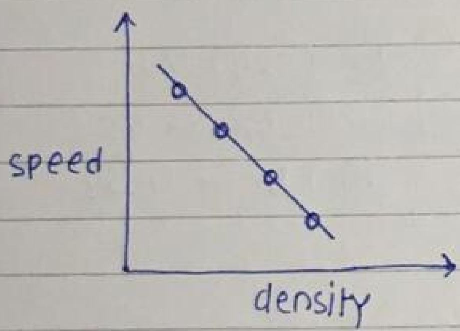
Que. Observation length is 0.5 km and the speed of Test vehicle is 20 KMPH. For the Data given below calculate Avg. speed of traffic stream, traffic flow & Traffic density. Also plot speed vs Density Relation, speed vs Flow and Flow vs Density Relation.

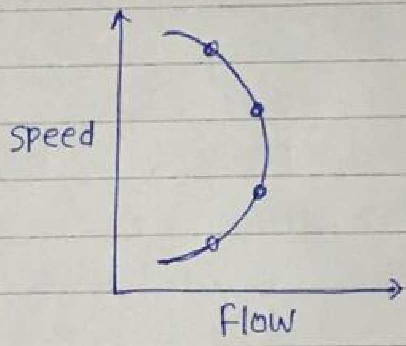
Sample	X	a	b	Y=a-b	$q=\frac{X+Y}{t_w+t_a}$ (veh/hr)	$\bar{t}=t_w-Y/q$ (hr)	$\bar{V}=l/\bar{t}$ (km/h)	$K=q/v$ (veh/km)
1	107	10	74	-64	860	0.099	5.05	170.29
2	113	25	41	-16	1940	0.033	15.15	128.052
3	30	15	5	10	800	0.0125	40	20
4	79	18	9	9	1760	0.0198	25.25	69.70

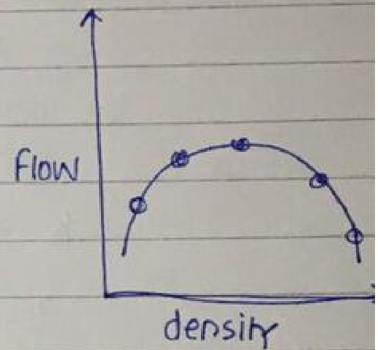
$l = 0.5$ km

$V_w = V_a = 20$ 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. $\quad V_w t_w = 5$ km $\qquad Y = 45 + 60 = 105$

$q = 45 \times 4 = 180$

in 15 min → 45

in 1 hr → 45x4

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

$$k = 3 \text{ veh/km}$$

$$q = kV$$

$$180 = 3V \Rightarrow V = 60 \text{ km/h}$$ 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 $\boxed{\text{Veh/Hr}}$ or pcu /hr

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

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 $\boxed{\text{Veh/km}}$

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

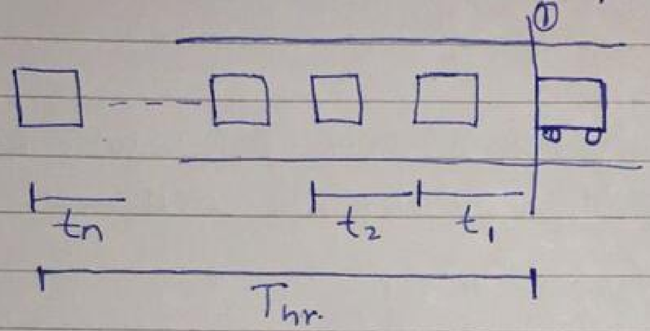
i.e *** $\boxed{q = KV}$ ***

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

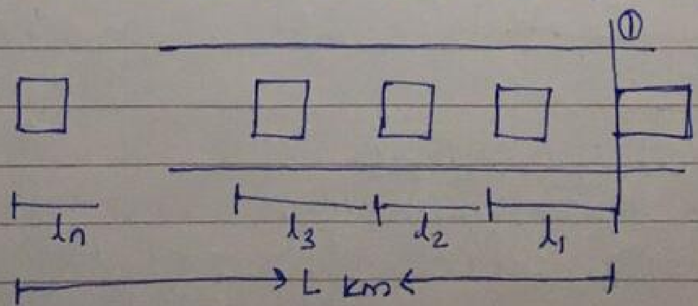
Hence no. of vehicles crossing secⁿ ① per hour (q) = $\frac{n}{T}$

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

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

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 $(K) = n/L$

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

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

$K \rightarrow$ 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 havel 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

$$q = KV$$

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

$$C_{\text{veh/hr}} = \frac{V^{\text{km/hr}}}{(\text{Avg. space Headway})_{\text{Min}}\ \text{km/veh}}$$

*

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

S → meter/veh.

C → capacity → veh/hr.

V → avg. Traffic speed in km/hr

S → Min. space headway (m/veh.)

① $S = 0.7\,V_b + L \Rightarrow \begin{cases} V_b \to \text{m/sec} \\ L \to 6\text{m if not given} \end{cases}$

② $S = SSD + L \to \begin{cases} \mu = 0.35 \to 0.4 \\ t_r = 2.5 \text{ sec.} \end{cases}$

ii) from Time headway

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

$$C_{\text{veh/hr}} = \frac{1}{(\text{Avg. Time headway})_{\text{Min.}}\ \text{hr/veh}}$$

*

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

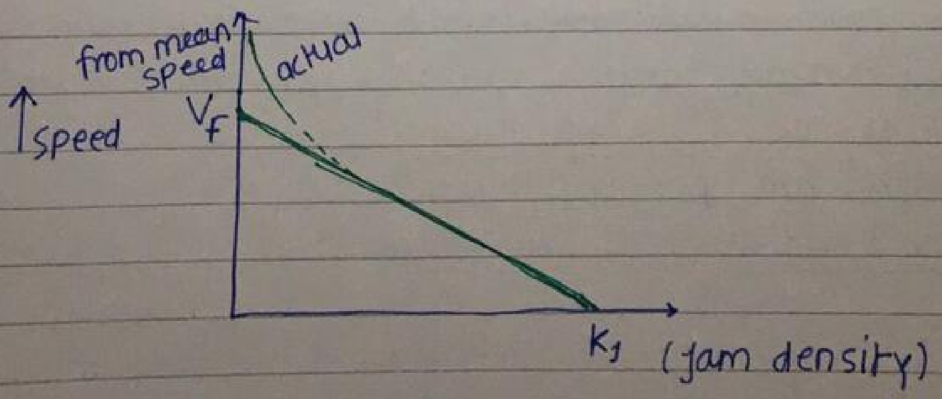
C → capacity → veh/hr

H_t → Min. Time headway in $\frac{s}{\text{veh}}$.

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 $\frac{dq}{dv} = 0$

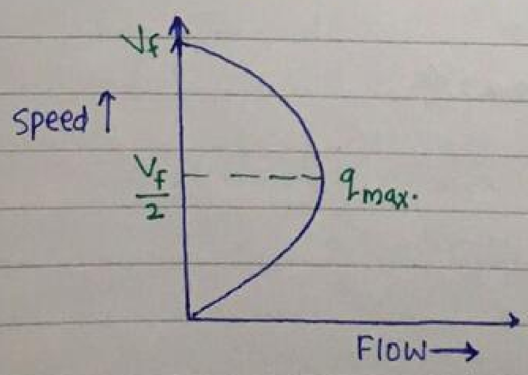
differentiating eq^n ① and substituting $\frac{dq}{dv} = 0$

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

substituting above value in eq^n ① 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 $\frac{dq}{dk} = 0$

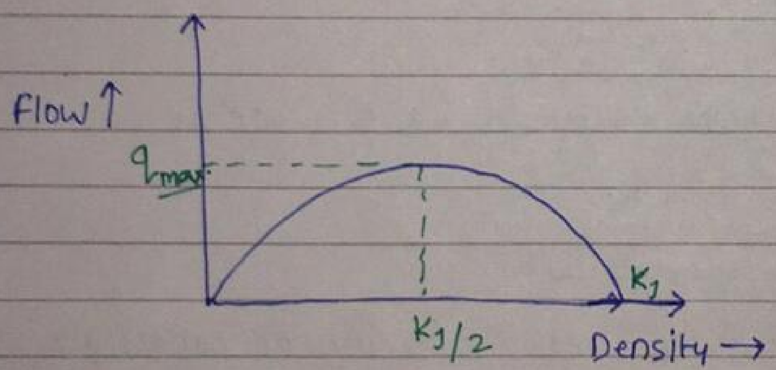
differentiating eq^n (ii) and subsbstituting $\frac{dq}{dk} = 0$
we've

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

substituting above value in eq^n (ii)

*

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



1) $C = \frac{1000\,V}{S}$ ~~$q_{max} \neq$~~ [illegible]

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?

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

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

$$q = KV_F\,(\ln K_J - \ln K)$$

$$q = KV_F \ln K_J - KV_F \ln K \quad -(i)$$

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

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

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

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

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

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

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

$$q = KV$$

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

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

$$q = 1000 \text{ Veh/hr}$$

for single lane

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

$$q = 500 \text{ Veh/hr}$$

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

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

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

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

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

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

for two-lanes

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

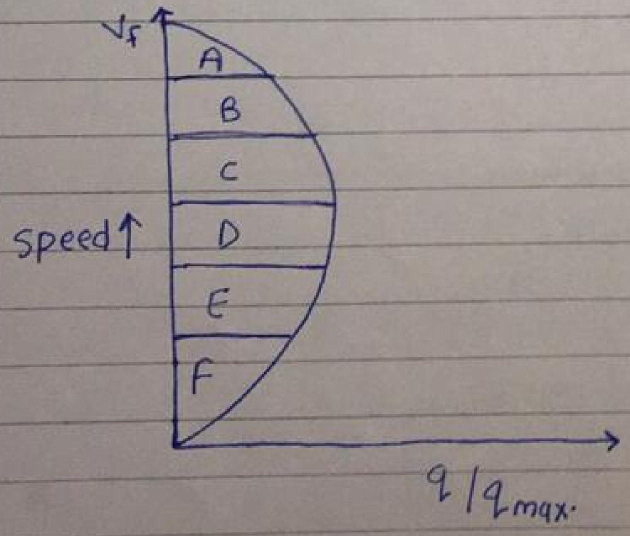
LEVEL OF SERVICE (LOS)

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 :-

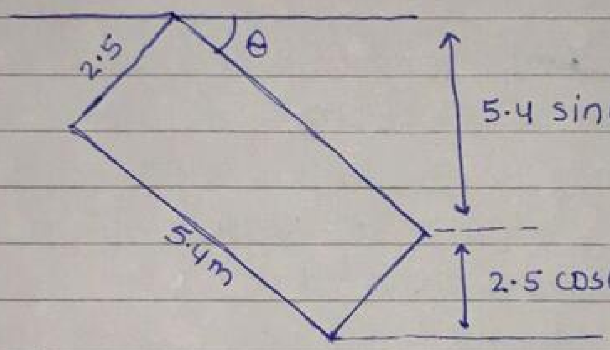
- used when there is width restriction
- least accident with main traffic
- parking Maneuver is difficult
- least no. of veh. per unit length of Road.

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 —

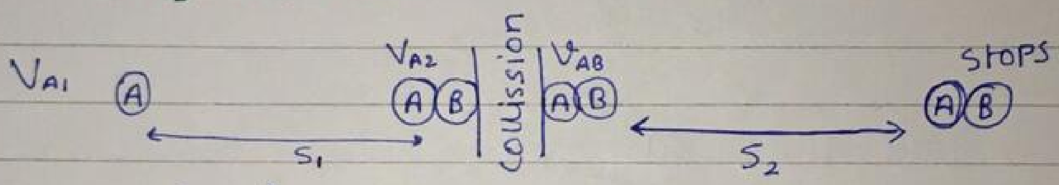
1) Location file :→ points of high accidents are Noted
2) Spot Map :→ Accident locations are shown in a Map
3) Condition Diagram :→ Drawings are prepared to scale showing all the imp. physical conditions at the time of accident
4) Collission Diagram :- Diagram showing path of vehicle and pedestrain involved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

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 collission is assummed.

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

<u>case-I</u> 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

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

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

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

$$V_{A2}^2 = V_{A1}^2 - 2\mu g S_1$$

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

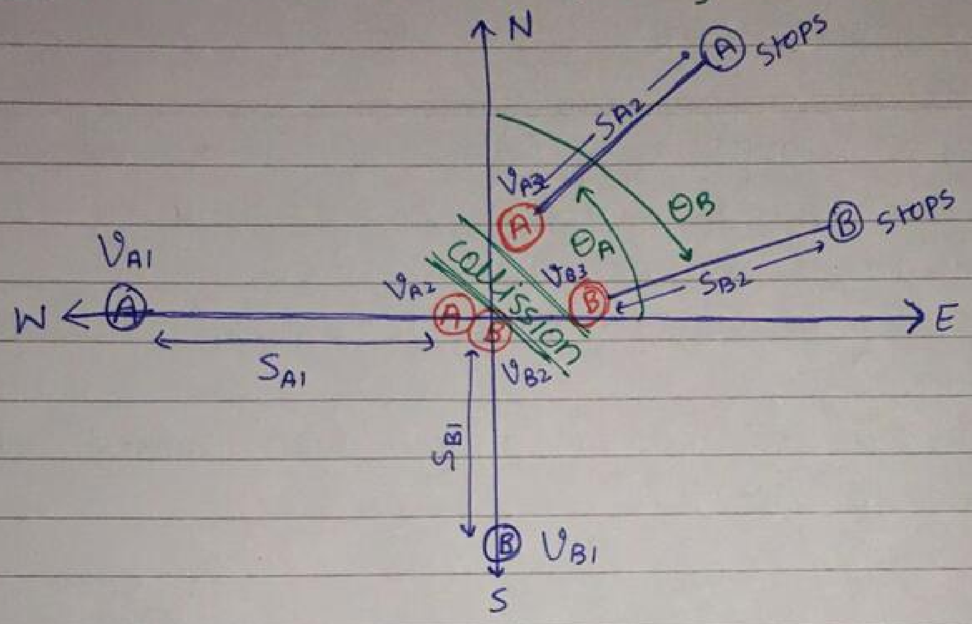
V_{A1} → initial speed of A. V_{A2} → speed of A just before collission

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:

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

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

For B:

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

From momentum conservation —

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

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

Y-axis

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

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

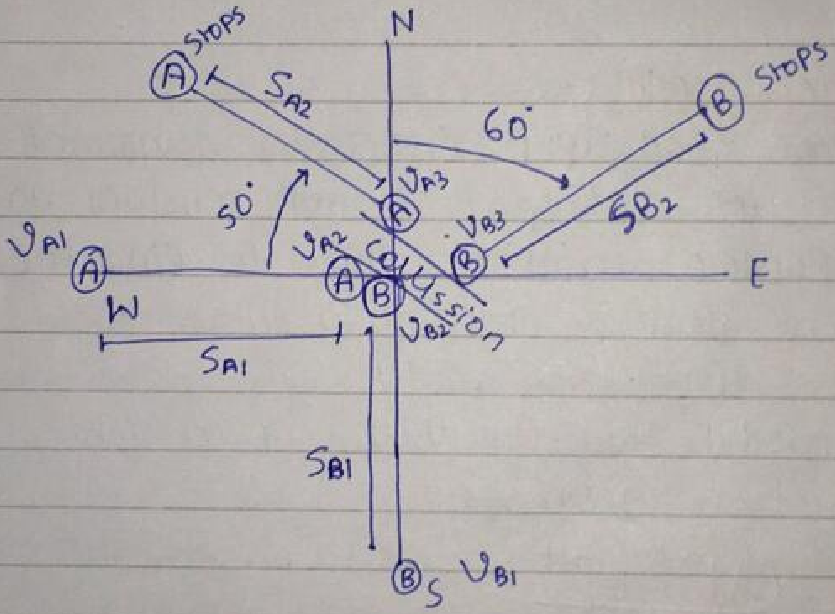
$$V_{A2}^2 = V_{A1}^2 - 2\mu g S_{A1}$$

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

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

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

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

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

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

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

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

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

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.

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

R/(a)17

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

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

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

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

$$P(3) = 0.18$$
$$P(4) = 0.09$$

$$P(0) + P(1) + P(2) + P(3) + P(4) + P(\geq 5) = 1$$
$$P(n \geq 5) = 1 - P(0) - P(1) - P(2) - P(3) - P(4)$$
$$P(n \geq 5) = 0.055$$

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

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

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

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:-

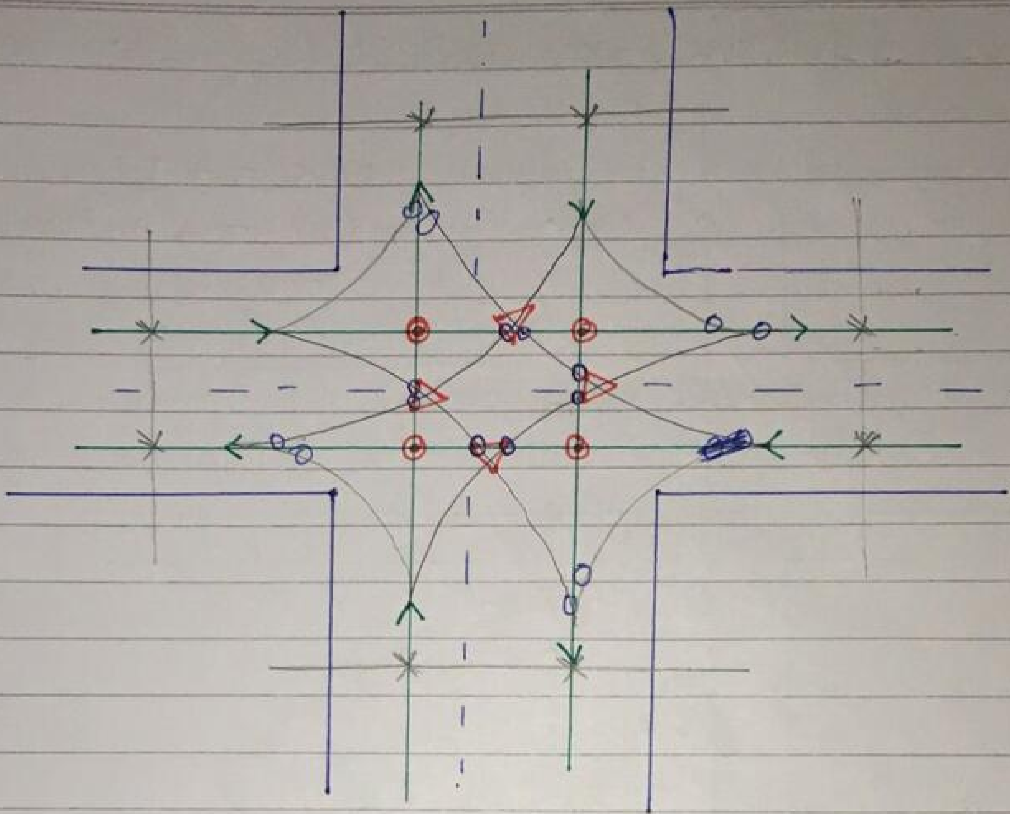
a) Through - Through crossing
b) Right Through crossing
c) Right - Right crossing

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



Major →

4 through - through crossing conflict

8 Right - through crossing conflict

4 Right - Right crossing conflict

Minor → 8 - merging conflict

Total = 16+8 = 24

pedestrain conflict = 8

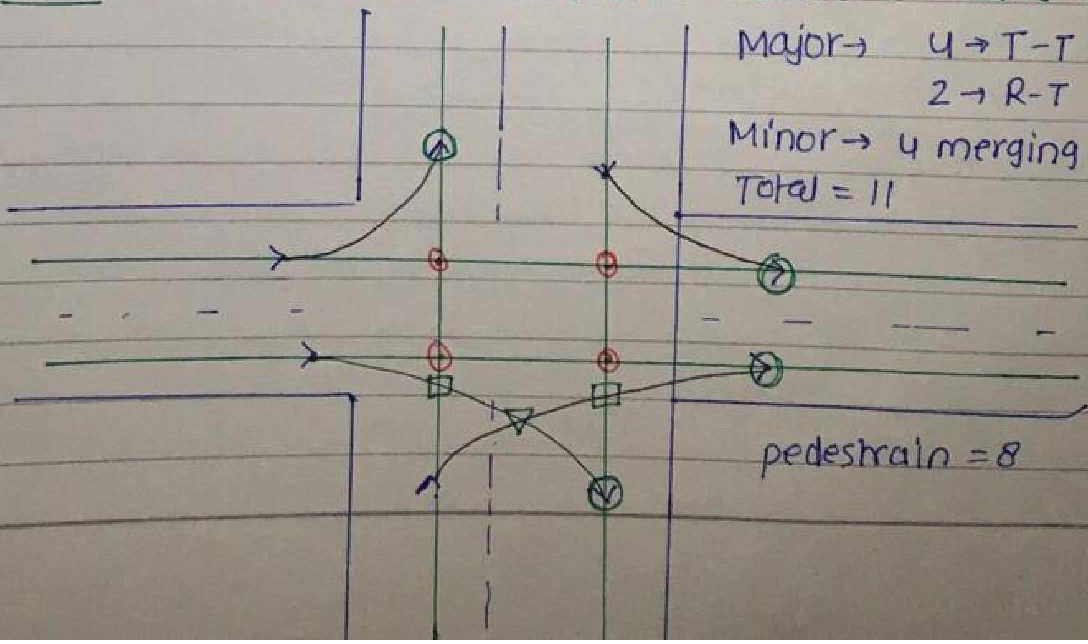
Case-II When 2-lane 2-Way Road crosses a 2-lane-1 Way Road.

Major → 4 → T-T

2 → R-T , 1 → R-R

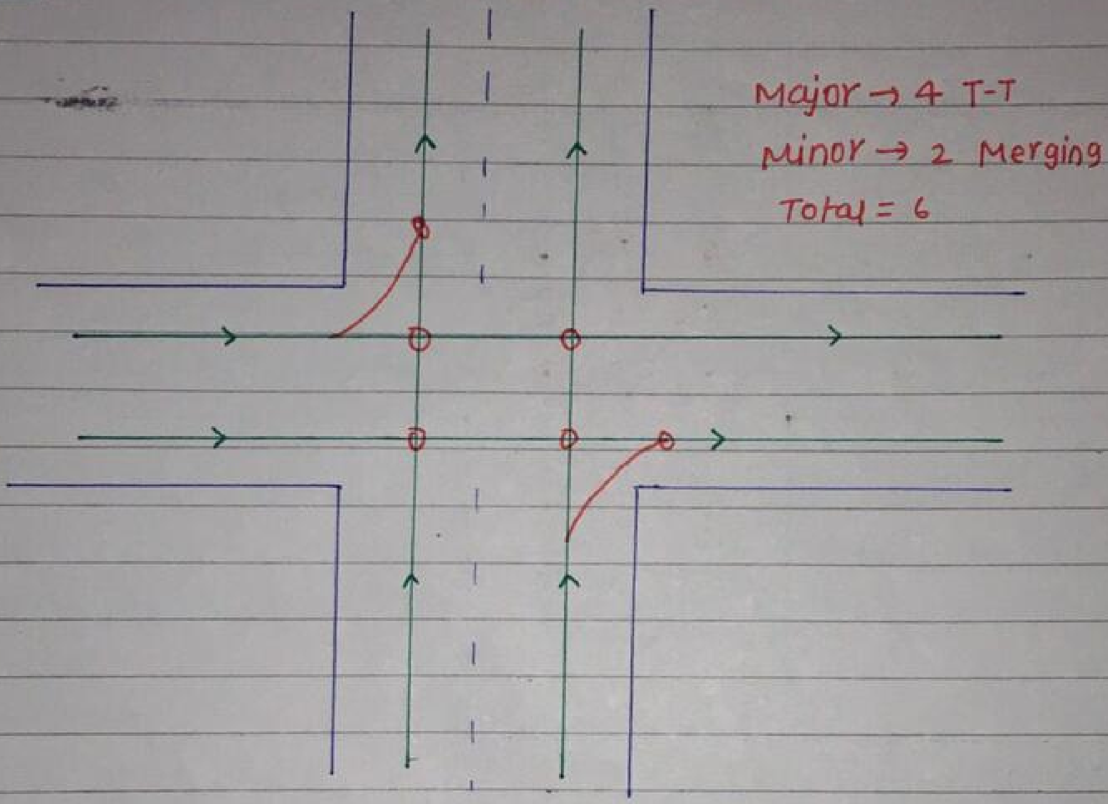
Minor → 4 merging

Total = 11



pedestrain = 8

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

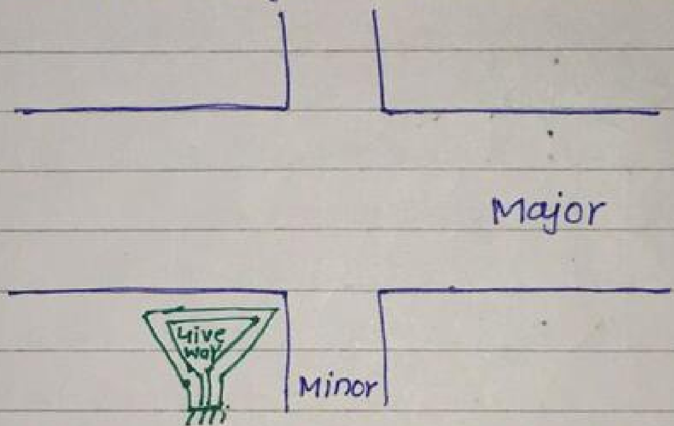


Intersection control

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.

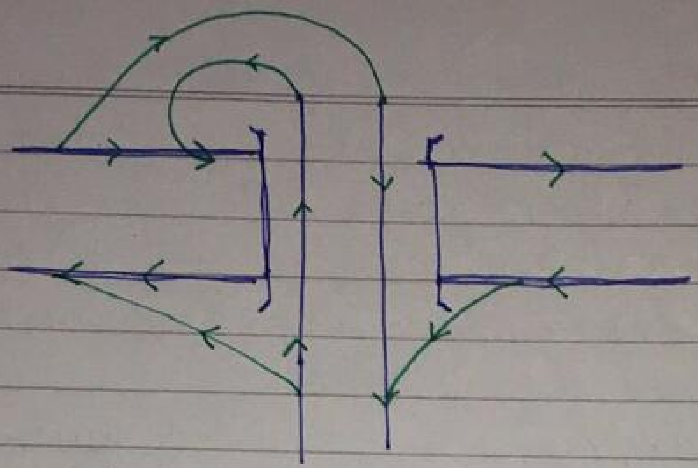
Types of Intersections :-

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

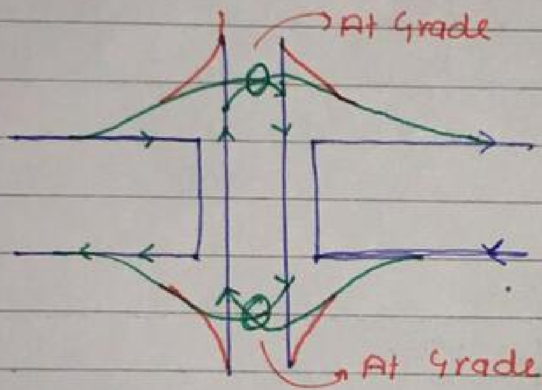
Interchange is a Grade seprated intersection with connecting roadways and ramps for turning traffic ~~on~~ b/w highway approaches

various types of interchanges are :-

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

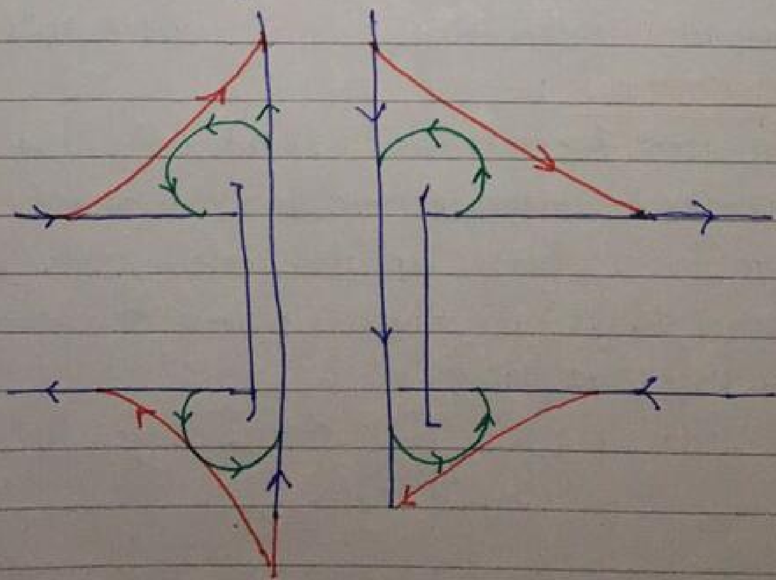


ii) Diomond interchange:- it is the simplest kind of 4 legged intersection. it is used when a Major Road crosses a Minor Road.



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

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

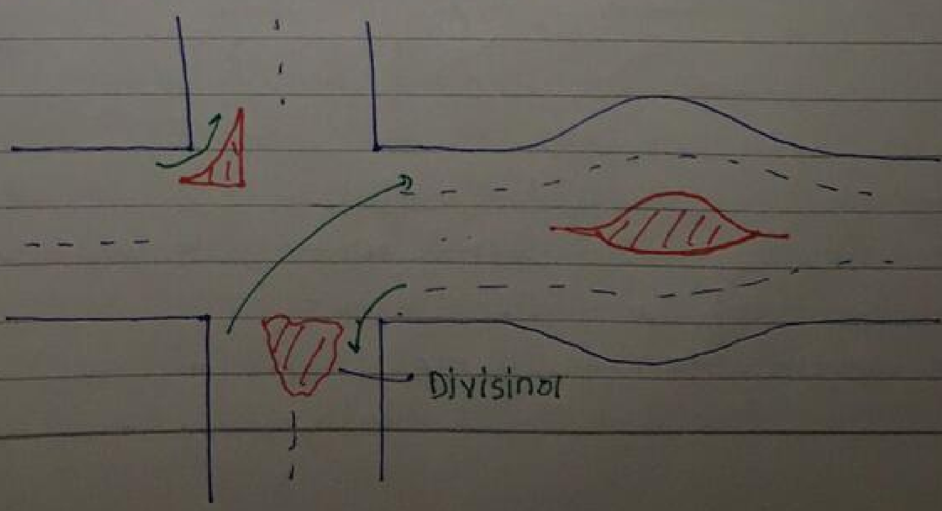


All crossing conflicts are eliminated and all mergings are left mergings
Area req. is more and is expensive

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

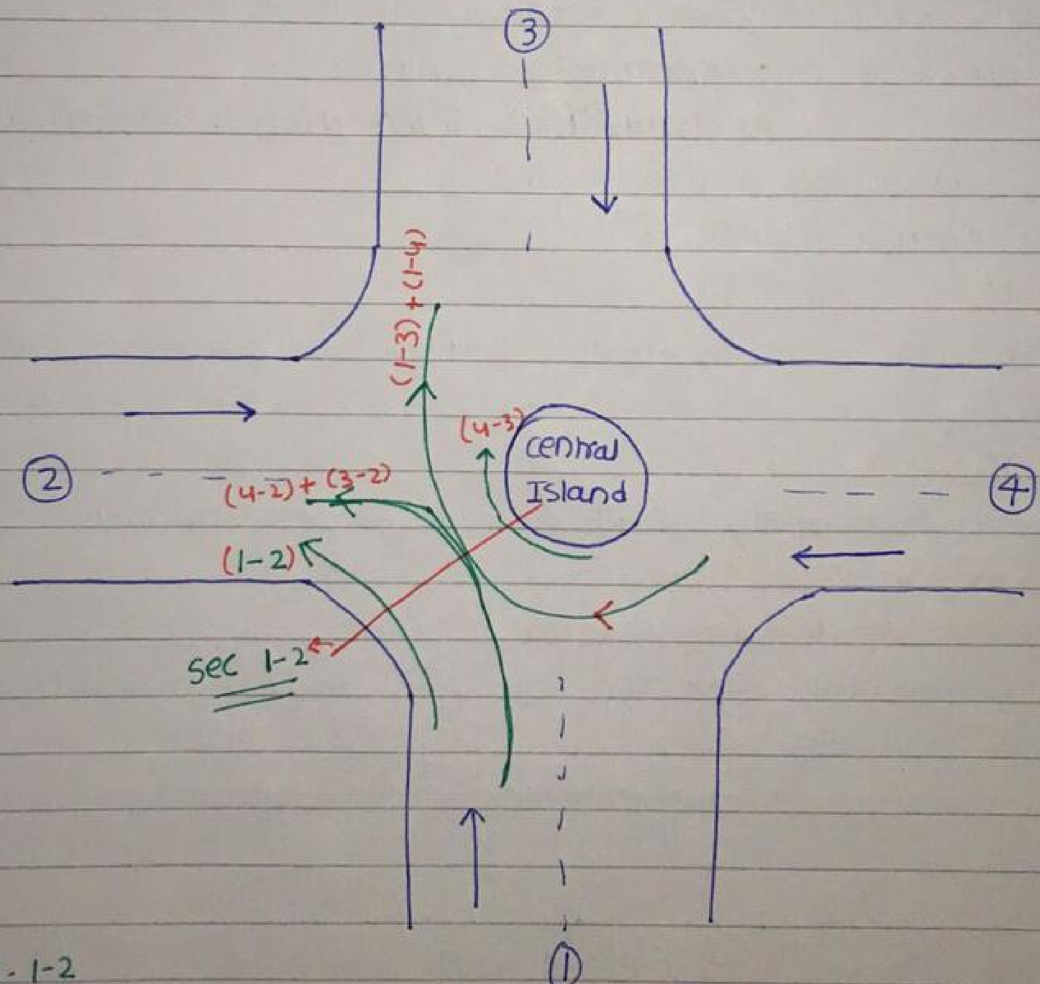
② At Grade intersection :-
All Roads of intersection meet at about same level.
It is classified as A) unchannelised intersection.
B) channelised intersection → channelization is done by providing traffic Islands.
various Traffic Ilands are
i) Divisional Island
It seprates opposite flow of traffic and 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 ~~ro~~ 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)

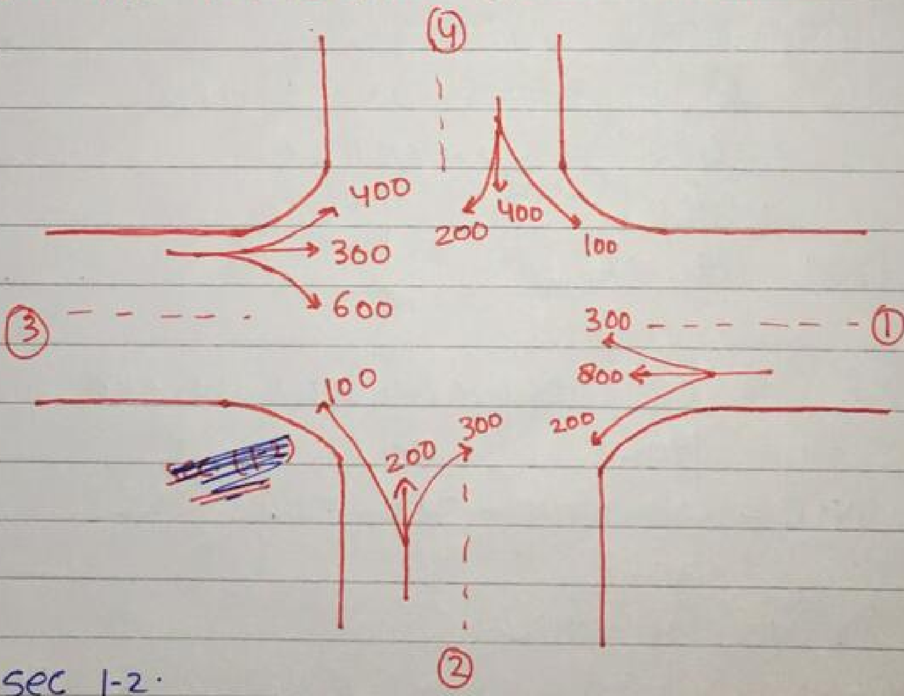
Weaving = (1-3) + (1-4) + (3-2) + (4-2)

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 severity of accidents.

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



sec 1-2.

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

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

$$N.W = 2500 - 2100 = 400 \text{ veh/hr}$$

sec^n 2-3

$T \cdot T = (2-1)+(2-3)+(2-4)+(4-3)+(1-2)+(1-3)$
$= 100+200+300+200+800+300$
$= 1900$

$W \cdot T = (2-1)+(2-4)+(1-3)+(4-3)$
$= 300+200+800+200$
$= 1500$

$N \cdot W = 400$ veh/hr

sec^n 3-4

$T \cdot T = (3-1)+(3-2)+(3-4)+(1-4)+(2-1)+(2-4)$
$= 400+300+600+300+300+200$
$= 2100$

$W \cdot T = (3-1)+(3-2)+(1-4)+(2-4)$
$= 600+300+300+200$
$= 1400$

$N \cdot W = 700$ veh/hr

sec^n 4-1

$T \cdot T = 200+400+100+300+300+600$
$= 1900$

$W \cdot T = 200+400+300+300$
$= 1200$

$N \cdot W = 700$ veh/hr

Advantages of Rotary:-

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

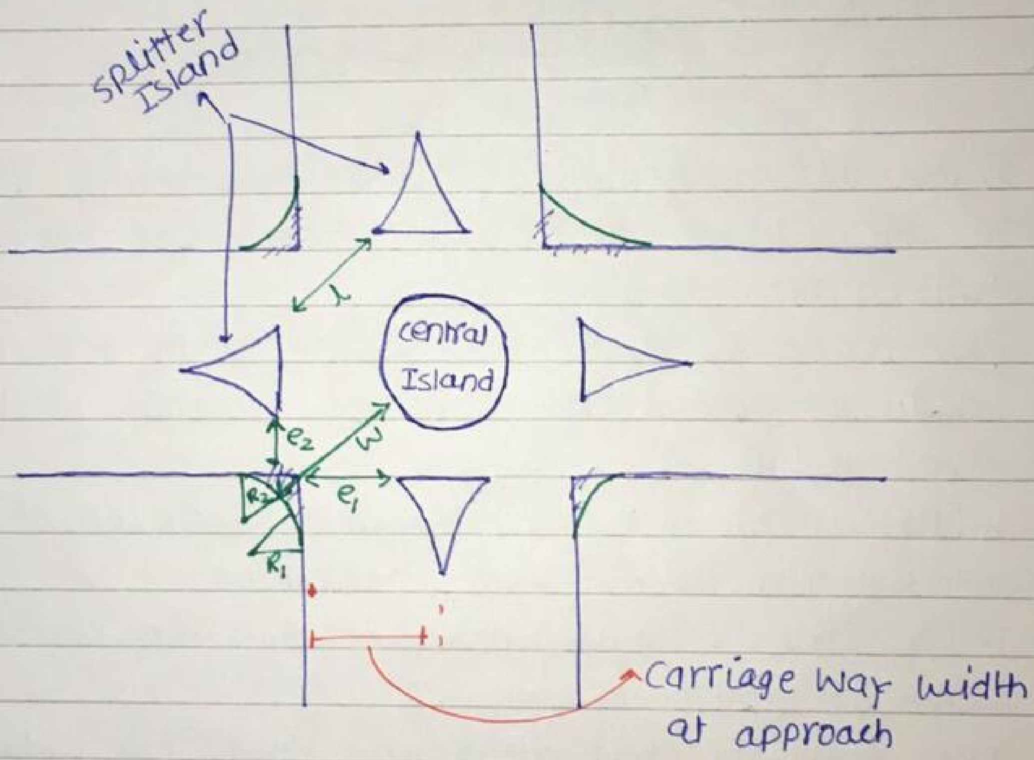
Disadvantages:-

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 preportion of Right turning ~~see~~ traffic is large (>30 %) Rotary becomes more benificial than signalized intersection

DESIGN ELIMENT OF ROTARY :-



$e_1 \rightarrow$ entry width
$e_2 \rightarrow$ exit width
$W \rightarrow$ width of weaving section
$l \rightarrow$ weaving length
$R_1 \rightarrow$ entry Radius
$R_2 \rightarrow$ 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 10m (URban) and 8m (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.

$$l = 4W$$

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

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.

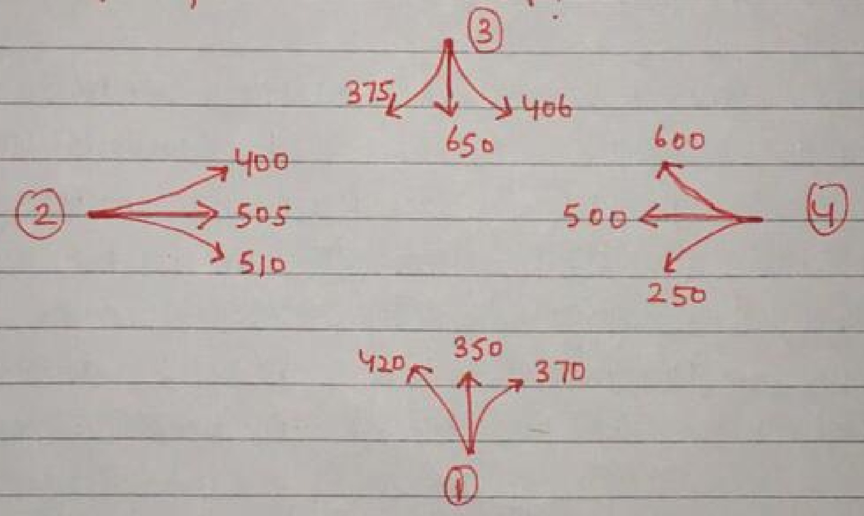
$$Q = \frac{280W\left(1+\frac{e}{W}\right)\left(1-\frac{p}{3}\right)}{\left(1+\frac{W}{l}\right)}$$

Q → capacity in weaving sec^n in pcu/hr.
W → weaving width
e → avg. of entry and exit width
l → wealing 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 10 m 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 $T \cdot T = 400 + 505 + 510 + 600 + 350 + 370 = 2735$

$W \cdot T = 505 + 510 + 350 + 600 = 1965$

p = Weaving Ratio = 0.718

sec- 3-4 $T \cdot T = 375 + 650 + 406 + 370 + 505 + 510 = 2816$

$W \cdot T = 375 + 650 + 370 + 505 = 1900$

p = Weaving Ratio = 0.675

sec- 4-1

$T \cdot T = 600 + 500 + 250 + 510 + 650 + 375 = 2885$

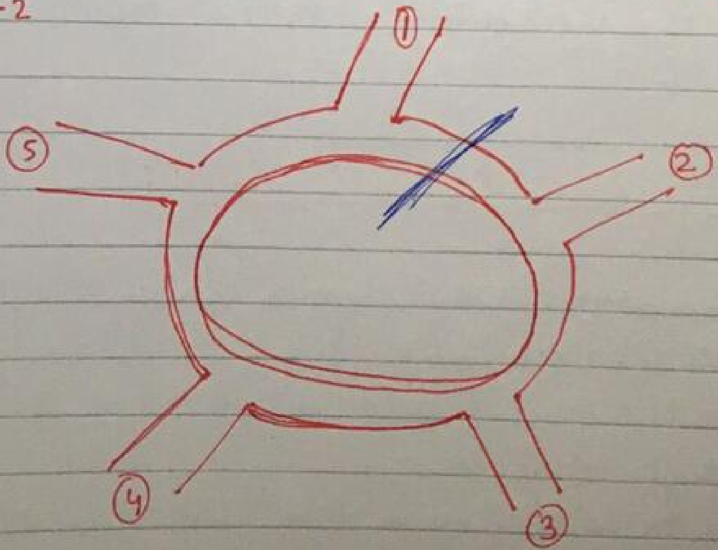
$W \cdot T = 500 + 600 + 510 + 650 = 2260$

$p = 0.783$

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

$Q = 3890.09$ 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



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$V_{1-2} = 37$	$V_{3-1} = 466$	$V_{4-1} = 180$	$V_{5-1} = 45$
$V_{1-3} = 303$	$V_{3-2} = 122$	$V_{4-2} = 54$	$V_{5-2} = 132$
$V_{1-4} = 64$	$V_{3-4} = 47$	$V_{4-3} = 18$	$V_{5-3} = 62$
$V_{1-5} = 52$	V_{3-5} 657	$V_{4-5} = 116$	$V_{5-4} = 15$

Secⁿ 1-2

$$T \cdot T = 37 + 303 + 64 + 52 + 122 + 18 + 54 + 132 + 62 + 15$$

$$T \cdot T = 859 \text{ veh/hr}$$

$$W \cdot T = 303 + 64 + 52 + 122 + 54 + 132$$

$$W \cdot T = 727 \text{ veh/hr}$$

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

Traffic Signals :- Traffic signals are classified as

1) Fixed Time signal or pre time signal :-
   signal timing is independent of traffic volᵐ (in real time)
2) semi-actuated signal :-
   signal timing is influenced by traffic in some dirⁿ 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 :-

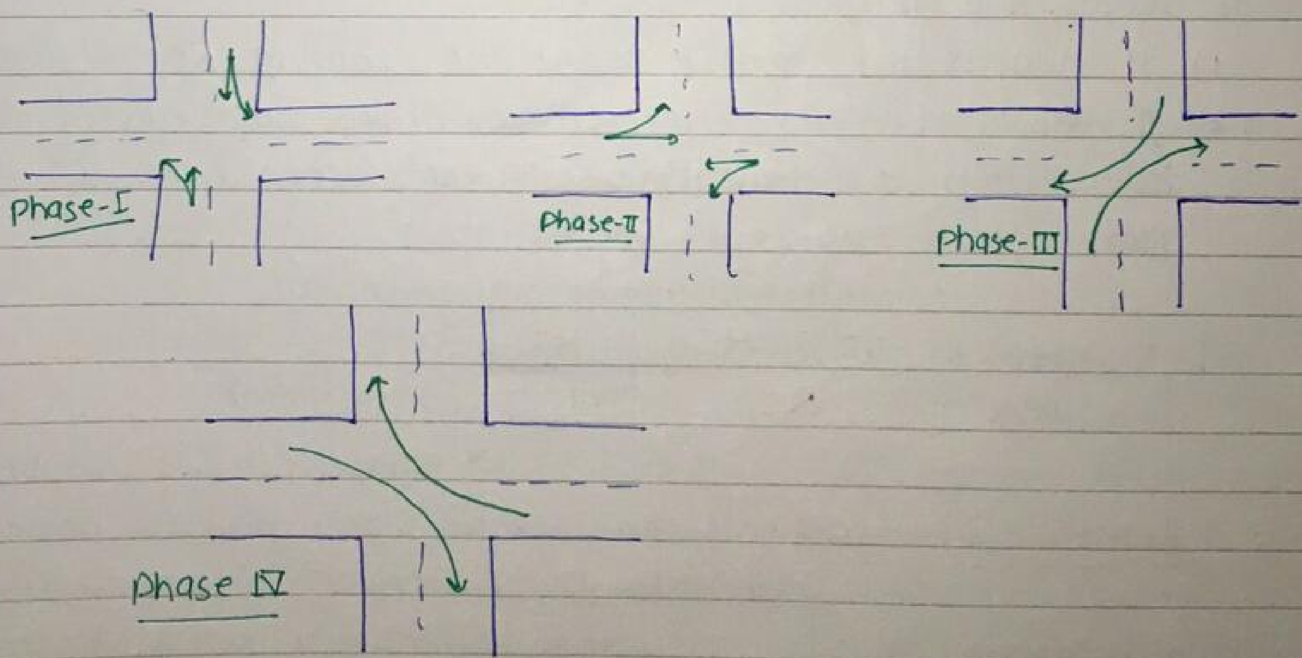
A time shedule is made to permit as nearly as possible a contineous operation of Group of vehicles along the Main Road at a reasonable speed.

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.

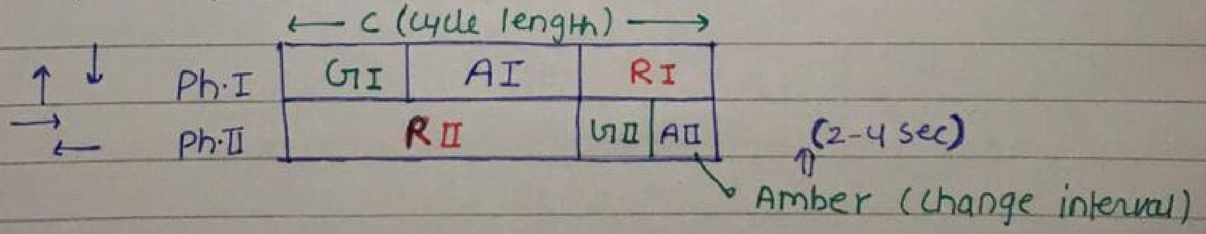
Intervals :- Intervals represents change from one stage to other. There are two types of intervals.

a) change interval
b) clearence interval

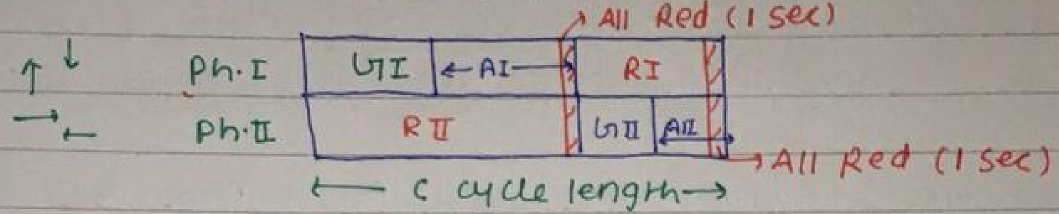
a) change interval :- It is also known as yellow time clearance amber or simply amber

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.

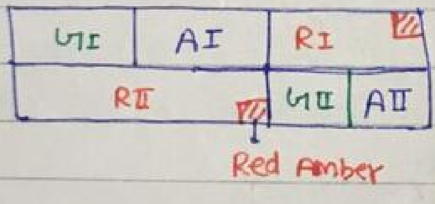


b) clearance interval :- clearance interval is also known as all Red interval. it is included after each Yellow interval and it indicates a period during Which all signal phases show Red signal. It is used for clearing the vehicle from intersection. All Red interval is optional and Generally provided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

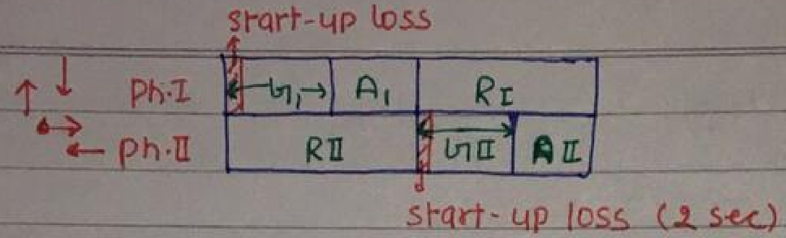


GREEN TIME :- The actual duration for Which green signal is turned on called as Green time

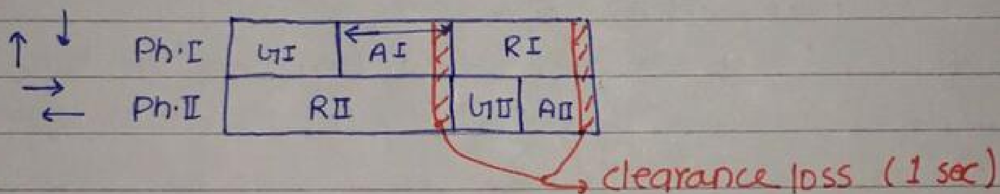
RED TIME :- The actual duration for Which Red signal is turned on called as Red time

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

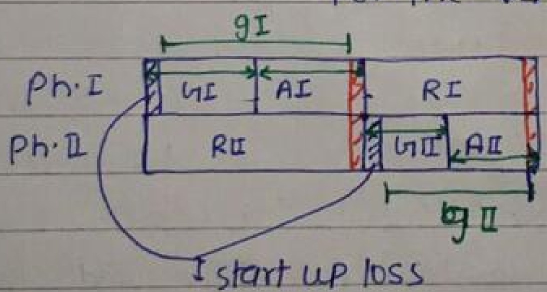
a) start-up loss : When indication changes from Red to Green driver's Don't start instantly. it is due to the reaction time of driver and is accounted for in start-up loss



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 = $\boxed{\frac{3600}{h \times c} \times g}$

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

Where $h \rightarrow$ Saturation time headway

(that is time headway a contineous movement of traffic)

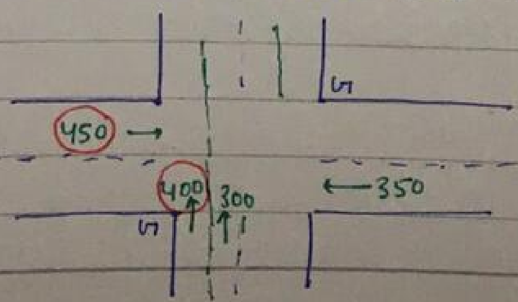
$\frac{g}{c} \rightarrow$ called 'Green Ratio'

where $g \rightarrow$ eff. Green time in a perticular phase in 1 cycle

$c \rightarrow$ 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

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

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

V_c = sum of critical lane Vol. of all Phases

$V_{ci} \rightarrow$ 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

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

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

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

$C \rightarrow$ 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

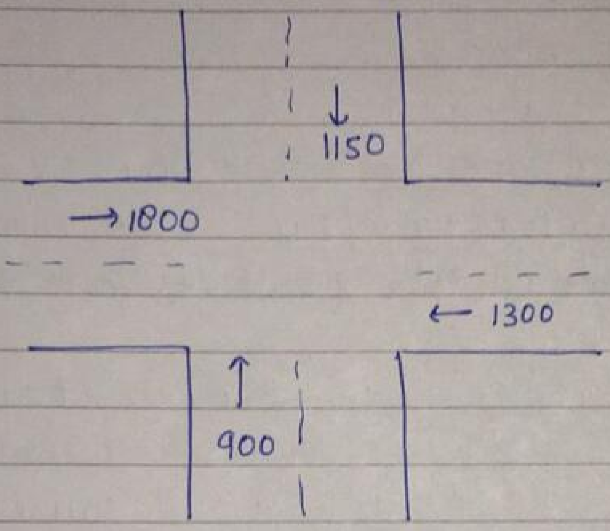
Que. Cycle time of an intersection is 60 sec., Green time is 27 sec. Amber time = 4 sec. Saturation headway is 2.4 sec/veh. Startup loss time = 2 sec. and clearance loss time = 1 sec. Find the capacity of movement / lane.

$$\text{capacity} = \frac{3600}{h} \times \frac{g}{c}$$

$$g = G + A - (\text{startup} + \text{clearance})$$
$$= 27 + 4 - (2+1)$$
$$= 28 \text{ sec.}$$

$$\text{capacity} = \frac{3600}{2.4} \times \frac{28}{60} = 700 \text{ veh/hr.}$$

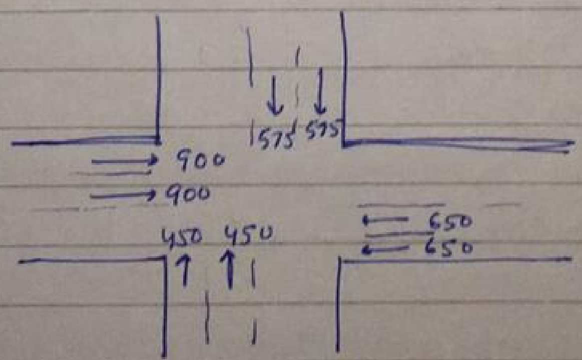
Que: Calculate cycle time for a two phase signal system for the intersection shown below:-
start up loss = 3 sec.
saturation time headway = 2·3 sec.



$V_{cI} = 1150$, $V_{cII} = 1800$

$$\sum_{i=1}^{2} \frac{V_{ci}}{S_i} = \frac{1150}{3600/2.3} + \frac{1800}{3600/2.3} = 1.88$$ (Not acceptable)

So, increase the no. of lanes.



$V_{cI} = 575$, $V_{cII} = 900$

$$\sum_{i=1}^{2} \frac{V_{ci}}{S_i} = \frac{575}{3600/2.3} + \frac{900}{3600/2.3} = 0.9423$$

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$$C = \frac{\Sigma t_{Li}}{1 - \Sigma \frac{V_{ci}}{S_i}} = \frac{2 \times 3}{1 - 0.9423} = 104.11 \text{ sec.}$$

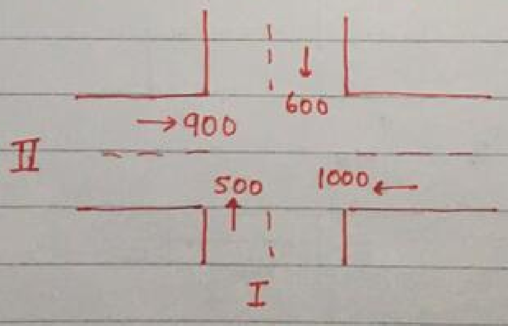
Adopt $C = \underline{\underline{105}}$ sec.

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

$$\text{Eff. Green time in 1 cycle} = \text{cycle time} - \text{loss in 1 cycle}$$
$$= c - \sum_{1}^{n} t_{Li}$$

$$\text{Eff. Green of } i^{th} \text{ phase } g_i = \left(c - \sum_{1}^{n} t_{Li}\right) \frac{V_{ci}}{\sum_{i=1}^{n} V_{ci}}$$

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.

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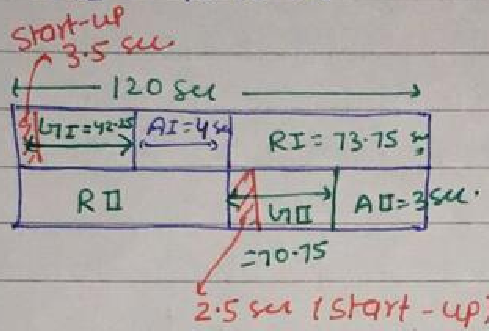
$$g_I = G_I + A_I - t_{sI}$$

$$42.75 = G_I + 4 - 3.5$$

$$G_I = 42.25 \text{ sec.}$$

$$g_{II} = G_{II} + A_{II} - t_{sII}$$

$$G_{II} = 70.75 \text{ sec.}$$

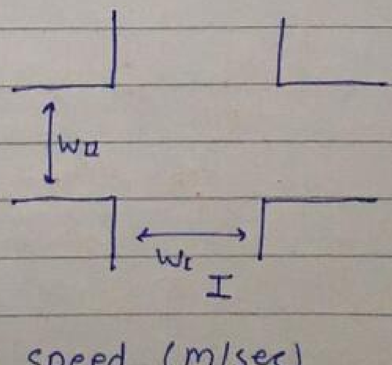


pedestrain crossing Requirement :-

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 →



$$G_{PI} = 7 + \frac{W_I}{1.2}$$

walk time

clearance time

$$G_{PII} = 7 + \frac{W_{II}}{1.2}$$

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.

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$$G_{PI} = R_I = G_{II} + A_{II}$$

$$G_{II} = G_{PI} - A_{II}$$

similarly $G_I = G_{PII} - A_I$

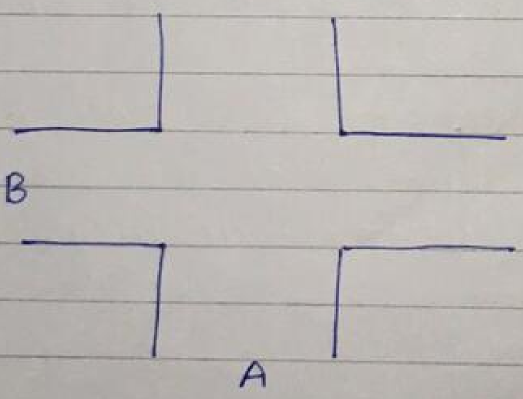
Design Methods for traffic signals.

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

$$x_A = \frac{n_A C}{15 \times 60}$$

$$x_B = \frac{n_B \times C}{15 \times 60}$$

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Green times for phase A & B is calculated as

$$G_A = x_A h$$
$$G_B = x_B h$$

$h \rightarrow$ 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

$$G_A = x_A h = 22.25 \text{ sec}$$
$$G_B = x_B h = 17.75 \text{ sec}$$

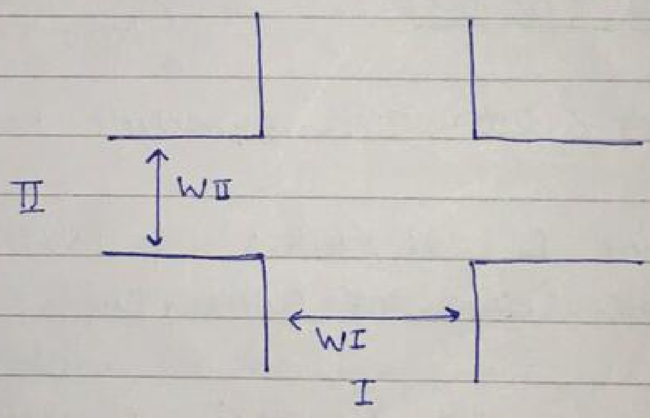
Final cycle time

$$C_f = x_A h + A_A + x_B h + A_B$$
$$= 45 \text{ sec.}$$

$C_f = C$ (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

$$\frac{G_{veh\,I}}{G_{veh\,II}} = \frac{V_{cI}}{V_{cII}}$$

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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 $< G_I \rightarrow$ 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 $\dfrac{G_{veh\,I}}{G_{veh\,II}} = \dfrac{V_{cI}}{V_{cII}} \Rightarrow$

$$G_{II} = 24.5 - 5 = 19.5$$

$$G_{I} = 19.5 - 5 = 14.5$$

putting $G_{veh\,I} = G_I$

$$G_{veh\,II} = \frac{G_I \times V_{cII}}{V_{cI}} = \frac{14.5 \times 260}{350} = \cancel{11.18}\ 10.7 \text{ sec.}$$

$G_{veh\,II} < G_{II}$ so, Not acceptable

$$\frac{G_{veh\,I}}{19.5} = \frac{350}{260} \Rightarrow G_{veh\,I} = 26.25 \text{ sec.} > G_I$$

Acceptable

Green times are taken as

$$G_I = 26.25 \text{ sec.}$$

$$G_{II} = 19.5 \text{ sec.}$$

$$C = G_I + A_I + G_{II} + A_{II}$$

$$= 55.75 \text{ sec.}$$

Adopt $C = 60$ sec

The additional cycle time $= (\Delta C = 4.25 \text{ sec})$ will be distributed in the Green time of both phases in proportion to their critical Lane volume

$$\Delta G_I = 4.25 \times \frac{350}{350+260} = 2.44 \text{ sec.}$$

$$\Delta G_{II} = 1.82 \text{ sec.}$$

Final Green times are $G_I = 26.25 + 2.44 = 28.69 \simeq 29$ sec.

$G_{II} = 19.5 + 1.82 = 21.32 \simeq 21$ 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 = $W_C/1.2$ = 17.5
	Ph IV	W.T = 21.5 sec	C.T = $W_{II}/1.2$ = 12.5 sec.	Don't Walk	

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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 —

$$\boxed{C = \frac{1.5L + 5}{1 - Y}}$$

$C \rightarrow$ cycle time in sec.

$L \rightarrow$ Total loss in one cycle

$$\boxed{L = nl + R}$$

$n \rightarrow$ no. of phases

$l \rightarrow$ start-up loss (Assume 2 sec. if Not given)

$R \rightarrow$ All Red loss

$Y \rightarrow$ critical Flow Ratio

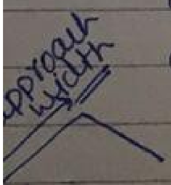
$$\boxed{Y = \sum_{i=1}^{n} \frac{V_{ci}}{S_i}}$$

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

$S_i \rightarrow$ saturation Flow / lane for i^{th} phase $\left(\frac{3600}{h_i}\right)$

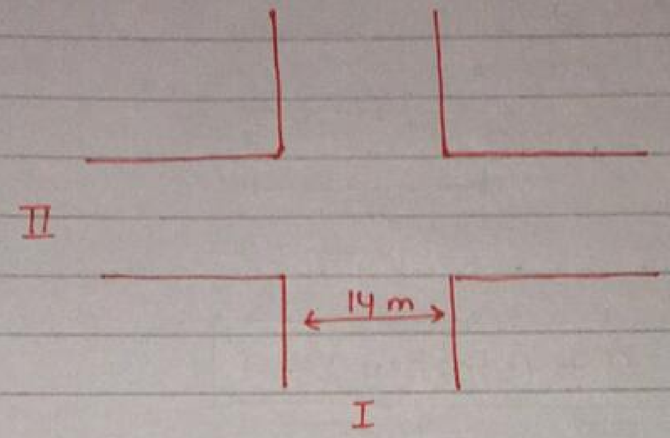
$h_i \rightarrow$ saturation time headway for i^{th} phase (Assume 2.5 sec if not given)

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

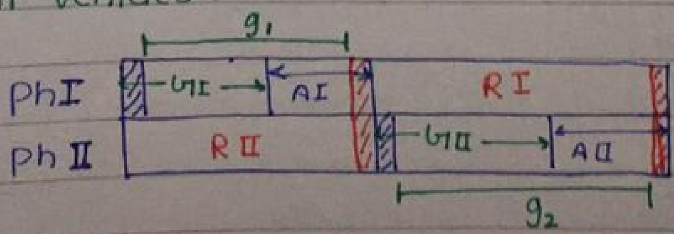
Eff Green time in 1 cycle = $C - L$

Effective Green time for i^{th} phase is calculated as

$$\boxed{g_i = (C-L)\,\frac{\left(\frac{V_{ci}}{S_i}\right)}{\Sigma\left(\frac{V_{ci}}{S_i}\right)}}$$

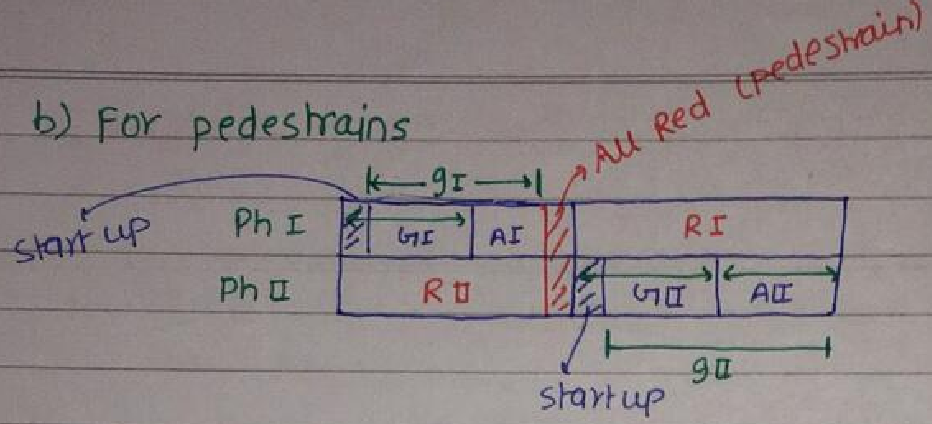
Types of All RED :-

A) For vehicles :-



$$\boxed{G = g + (\text{start upp} + \underset{\text{(all red)}}{\text{clearance}}) - A}$$

b) For pedestrains



$$\boxed{G = g + (\text{start up}) - A}$$

Que. Design a two phase signal system for the Data given below using Webster's Method.

$q_I = 400$ Ped/hr

$q_{II} = 250$ pcu/hr

$S_I = 1250$ pcu/hr

$S_{II} = 1000$ pcu/hr

All red for pedestrain = 12 sec.

2 phases $\Rightarrow n = 2$

$$Y = \frac{V_{cI}}{S_I} + \frac{V_{cII}}{S_{II}}$$

$$= \frac{400}{1250} + \frac{250}{1000}$$

$$= 0.32 + 0.25$$

$$= 0.57$$

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.

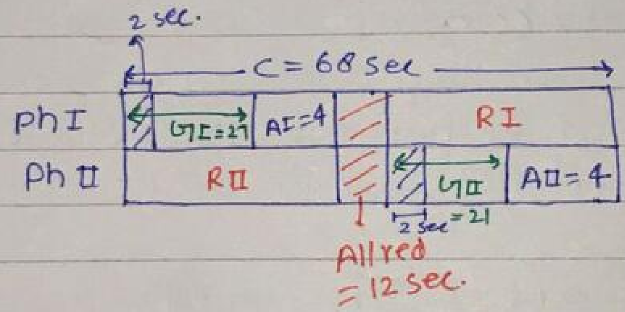
Eff. green for phase1 $= 52 \times \frac{0.32}{0.57} = 29.19 = 29$ sec

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.

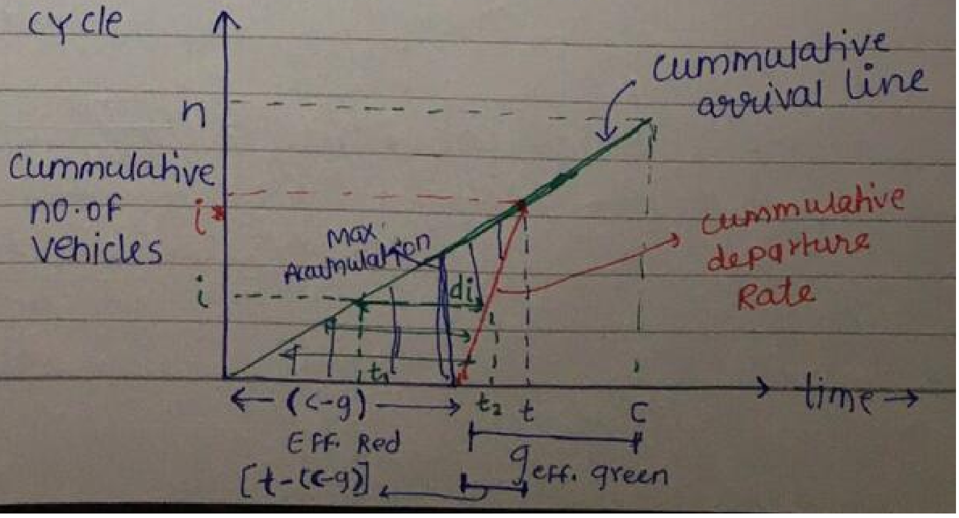
Assuming Amber = 4 sec/pha



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



$c \rightarrow$ is the cycle time

$g \rightarrow$ eff. Green time

$d_i \rightarrow$ delay for the i^{th} vehicle $(t_2 - t_1)$

$V \rightarrow$ slop of cummulative arrival line i.e Uniform Rate of arrival.

$S \rightarrow$ 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 eq^n 1 & 2

$$\text{Total Delay} = \frac{1}{2} \times (c-g)^2 \cdot \frac{SV}{S-V} = \frac{SV(c-g)^2}{2(S-V)}$$

$$\text{Avg. Delay per Vehicle} = \frac{\text{Total Delay}}{\text{Total no. of Vehicles in one cycle}}$$

$$= \frac{SV(c-g)^2}{2(S-V) \times Vc}$$

Scanned with CamScanner

$$\text{Avg. Delay per Vehicle} = \frac{c(1-g/c)^2}{2(1-V/s)}$$

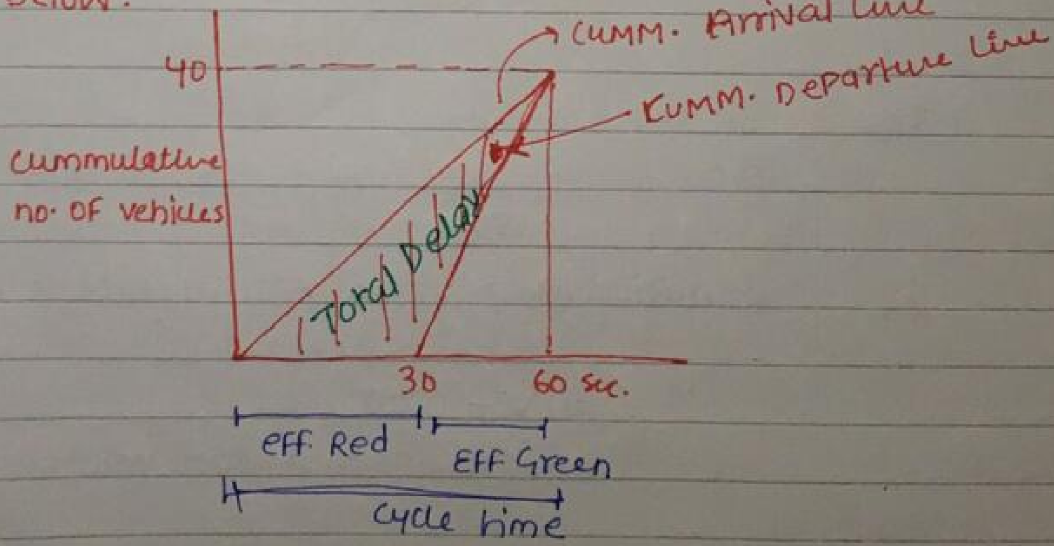
① Horizontal ordinate b/w cummulative arrival and cummulative departure line represents delay.

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

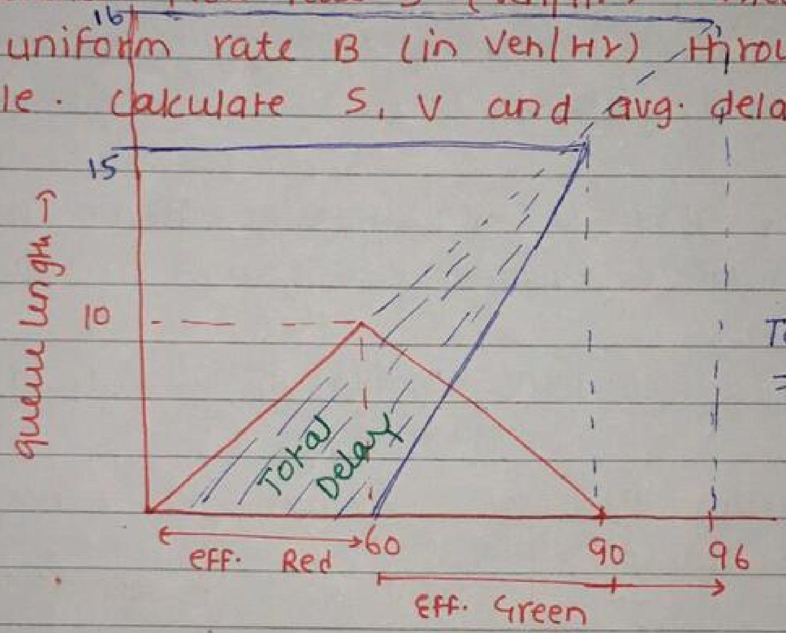
Que. calculate the avg. delay per vehicle for the figure shown below.



$$\text{Avg. delay per veh} = \frac{\frac{1}{2} \times 30 \times 40}{40}$$

$= 15$ 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. $= \dfrac{\frac{c}{2}\left(1 - g/c\right)^2}{(1 - V/S)}$

$= 28.125$ 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

$$(PHF)_{15} = \frac{V_{60} \rightarrow \text{60 Min Vol}^m \text{ in peak hr.}}{4 \times V_{15} \rightarrow \text{peak 15 Min vol}^m \text{ in that hour.}}$$

$$\boxed{0.25 \leq (PHF)_{15} \leq 1}$$

1 → incase of uniform Arriva

0.25 → in case of Max. variation

$$(PHF)_{20} = \frac{V_{60}}{3 \times V_{20}} \rightarrow \quad 0.33 \leq (PHF)_{20} \leq 1$$