CHAPTER 4

Traffic Engineering

INTRODUCTION

Traffic engineering is defined as that phase of highway engineering which deals with the planning and design of highway and with the traffic operation there on for the safe convenient and economic transportation of persons and goods . This is achieved by systematic traffic studies, scientific analysis and engineering applications. Traffic studies are further divided into:

1. Traffic characteristics 2. Traffic studies and analysis 3. Traffic control regulation

4.1 Function of Traffic Engineering

The function of traffic engineering can be covered in the following five categories:

1. Traffic characteristics
   - (a) Vehicular limitations like the weight, size and power of the vehicle
   - (b) Road user limitation
     - (*i*) Physical limitations like vision, hearing, fatigue etc.
     - (*ii*) Mental limitations like intelligence, skill, experience of drivers etc.
     - (*iii*) Emotional limitations like attentiveness, impatience etc.
2. Traffic operations which constitutes the traffic regulations, traffic control devices like traffic signs, signals, marking etc.
3. Traffic geometric design involving the design of expressways, streets, interchanges, intersections, parking etc.
4. Traffic planning like programme of construction, offstreet parking etc.
5. Traffic administration

4.2 Traffic Characteristics

In traffic characteristics, we study about the road user characteristics and vehicular characteristics.

4.2.1 Road User Characteristics

Human element is involved in all the actions of road users i.e. motorist, pedestrian etc. Factors affecting road user characteristics are:

(i) Physical: Vision, hearing, strength and the general reaction to traffic situations.

NOTE: The temporary physical characteristics of the road users affecting their efficiency are fatigue, alcohol or drugs and illness.

(ii) Mental: Knowledge, skill, intelligence, experience etc.

(iii) Psychological: Attentiveness, fear, anger, superstition, impatience, general attitude towards traffic and regulations.

(iv) Environmental: Traffic stream characteristics, facilities to the traffic, atmospheric conditions and the locality.

4.2.2 Vehicular Characteristics

It is quite important to study the various characteristics of vehicle because a road can be designed for any vehicle but not for an indefinite vehicle. The various vehicular characteristics affecting the road design is classified as static and dynamic characteristics of the vehicle.

4.2.2.1 Static Characteristics of vehicle

Static characteristics of vehicles affecting road design are the dimensions, weight and minimum turning radius.

Maximum Dimensions of Road Vehicles:

(*i*) Maximum width of vehicle = 2.5 m

(*ii*) Maximum height: It affects the clearance of the overhead structures and visibility of driver
- (a) Single decked vehicle = 3. 80 m
- (b) Double decked vehicle = 4.75 m

(*iii*) Maximum Length: It affects the capacity, OSD and movability of vehicle
- (a) Single unit truck with two or more axles = 11.0 m
- (b) Single unit bus with two or more axles = 12.0 m
- (c) Semi trailer tractor combinations = 16.0 m
- (d) Tractor and trailor combinations = 18.0 m

NOTE: No vehicle is allowed to be of more than two units and no such combinations, laden or unladen is allowed tc have an overall length exceeding 18 m.

Weight of Vehicles:

(*i*) Maximum weight of loaded vehicle affects the design of pavement thickness and gradients.

(*ii*) No single axle load, as suggested by IRC, should exceed 102 kN (10.2 tonnes) and for tandom axle 180 kN (18 tonnes).

(*iii*) The gross load of any vehicle or combination of vehicles should not exceed the weight worked out by the following formula:

$$W = 1525\,(L + 7.3) - 14.7\,L^2$$

Where, W = The gross weight of the vehicle in kg

L = The distance in metre between the extreme axles measured parallel to the axis of the vehicle

NOTE: The above relation holds good only when 'L' is greater than 2.50 m.

Power of Vehicle:

The power of the heaviest vehicles and their loaded weights govern the permissible and limiting values of gradient on road. Limiting gradients are governed by both the weight and power of the heavy vehicles. Stability of vehicle and its safe movement on horizontal curve depends upon the width of vehicle and height of centre of gravity.

Minimum Turning Radius:

(*i*) It depends upon the length of wheel base and the features of the steering system.

(*ii*) It affects the design of sharp curves for the movement of vehicles at slow speed.

4.2.2.2 Dynamic Characteristics of Vehicles

Speed, acceleration and braking characteristics are the dynamic characteristics of a vehicle. The speed and acceleration of a vehicle depends upon the power of the engine and the resistance to overcome. The deceleration and braking characteristics of vehicles depend on the design and type of braking system and its efficiency.

Braking Test is conducted to measure the skid resistance of a pavement surface. Atleast two of the following three measurements are needed during braking test to find skid resistance.

(*i*) Braking Distance, L (Length of skid mark on the road surface in m)

(*ii*) Initial Speed, u (in m/s)

(*iii*) Actual duration of brake application , t (in seconds)

Following cases involved in the braking test are discussed below:

Case-1: When initial velocity and braking length is known

After the application of brake, workdone by the frictional force to stop the vehicle will be equal to the kinetic energy of the vehicle

$$\therefore \quad \frac{1}{2}mu^2 = f\text{WL} \Rightarrow \frac{1}{2}\frac{W}{g}u^2 = f\text{WL} \Rightarrow f = \frac{u^2}{2gL}$$

where, f = Coefficient of friction

g = Acceleration due to gravity (in m/s²)

Case-2 : When initial velocity and actual direction of brake application is measured

After application of brakes frictional force acting on the vehicle to stop it will be equal to the driving force of the vehicle.

$$\therefore \quad fmg = ma$$

$$\Rightarrow \quad f = \frac{a}{g}$$ (where, a = Retardation of vehicle during skidding in m/s²)

Case-3 : When braking length and actual duration of brake application is measured

On the application of brakes, the vehicle comes to rest with the retardation of 'a'.

Using equation of motion, $v^2 = u^2 + 2\,as$

As we know that, $v = 0$ and a is retardation

$$\therefore \quad 0 = u^2 - 2aL \qquad \ldots(i)$$

Now, $v = u + at \Rightarrow 0 = u - at$

$$\therefore \quad u = at \qquad \ldots(ii)$$

from equation (*i*) and (*ii*), we get

$$a^2t^2 = 2aL$$

$\therefore \quad a = \frac{2L}{t^2}$

and we also know that, $\quad f = \frac{a}{g}$

$\therefore \quad f = \frac{2L}{gt^2}$

Example-4.1 A vehicle travelling at the 80 kmph speed, stopped within 2.5 seconds after the application of the brakes. Determine the average skid resistance.

Solution:

Initial speed, $u = 80$ kmph or $\frac{80}{3.6} = 22.22$ m/s, Braking time = 2.5 seconds

using the equation of motion for uniform retardation

$$v = u + at$$

$\therefore$ Retardation, $\quad a = \frac{u}{t} = \frac{22.22}{2.5} = 8.88 \text{ m/s}^2 \quad (\because v = 0)$

Now, force, $\quad F = ma = \frac{Wa}{g} \quad \ldots(i)$

and, $\quad F = fW \quad \ldots(ii)$

From equation (*i*) and (*ii*), we get $\quad \frac{Wa}{g} = fW \Rightarrow f = \frac{a}{g} = \frac{8.88}{9.81} = 0.905$

Do youself: A vehicle was stopped in 2.0 seconds by fully jamming the brakes and the skid marks measured 8.0 m. Determine the average skid resistance. **(Ans. 0.407)**

4.3 Traffic studies and Analysis

Traffic studies or surveys are carried out to analyze the traffic characteristics. A detailed knowledge of the operating characteristics of the traffic is essential to form a basis for the establishment of traffic control or for design of highways.

The results of data collected are used in

(*i*) traffic planning
(*ii*) traffic management
(*iii*) economic studies
(*iv*) traffic and environmental control
(*v*) monitoring trends

Traffic studies are broadly classified into two categories:

(*i*) Those concerned with the characteristics of traffic in transit
- (a) Traffic volume study
- (b) Speed studies
- (c) Origin and destination study
- (d) Traffic flow characteristics and studies
- (e) Traffic capacity studies

(*ii*) Those related to land use movements
- (a) Parking Studies
- (b) Accident Studies

4.3.1 Traffic Volume Study

Volume of traffic is a very important variable and is essentially the quantity of movement per unit of time at a specified location. It is expressed as vehicles per day and vehicles per hour

Traffic volume, $\quad q = \frac{n \times 3600}{T}$

where, n = The number of vehicles passing a point in the roadway in 'T' seconds
q = Equivalent hourly flow

NOTE: Complete traffic volume study includes classified volume study (i.e. number of different of vehicle), directional study (distribution on different lanes).

Volume studies are basically useful to establish:

(*i*) the relative importance of any route
(*ii*) the fluctuations in flow
(*iii*) the distribution of traffic on the road system
(*iv*) the trends in the road use

Methods to Measure Traffic Volume

Specific methods of the various traffic volume studies are given below:

1. **Manual Counting:** In manual counts trained persons are posted at each lag of an intersection to count and record the number of trucks or buses, bullock carts, cars etc.
2. **Automatic Counters:** These are mainly of two types :
   (*i*) Permanent counting recorders
   - (a) Magnetic Detectors
   - (b) Pressure Sensitive Detectors
   - (c) Electronic Detectors

   (*ii*) Portable recorders which are smaller in size and actuated by air switches.

The main advantage of automatic counter is that it can work throughout the day and night for the desired period. Disadvantage associated with the automatic counter is that it is not possible to get traffic volumes of various classes of traffic in the stream and the details of turning movements.

Types of Volume Count

(i) Average Daily Traffic Volume (ADT): It is the average of 365 days. It can be used for the following purposes:
- (a) Planning major street
- (b) Improvement, construction or reconstruction of roads
- (c) Computing accident rates
- (d) Computing highway user revenue

(ii) Classified Volumes: It involves the composition of traffic i.e. trucks, cars, rickshaws, etc. These volumes can be used for geometric design of roads.

(iii) Hourly Volume: This type of volume count is used for:
- (a) Determining deficiency in capacities, geometric designs, etc.
- (b) Determining number and width of lanes
- (c) Parking demands
- (d) Planning traffic control (traffic signs, signals, timing of signals, etc.)
- (e) Location of interchanges

(iv) Pedestrian Volumes: These are utilized in planning the crosswalks and signals for pedestrians.

(v) Turning Movements Counts: These are used in the design of intersections and interchanges, planning of signal timings and turn prohibitions, channelization, etc.

Presentation of Traffic Volume Data

(i) Annual Average Daily Traffic (AADT): It is the average volume of traffic at a particular location over a full 365 days and it is given as:

$$AADT = \frac{\text{Total yearly traffic}}{365}$$

It helps in deciding the relative importance of a route and in phasing the road development program. If the flow is not measured for all the 365 days, but only for a few days then the average flow is known as Average Daily Traffic (ADT). For this minimum of seven days count is done to include the daily variation like on Saturday and Sunday.

NOTE: (i) While calculating ADT seasonal variation of traffic is not accounted.
(ii) Design hourly volume = Approximately 7 to 8% of AADT.

(ii) Trend Chart: They show the volume trends prepared over period of years. It helps in highway planning and design of pavement by the prediction of future loads. It helps in calculating the rate of growth of traffic which is used for calculation of design traffic volume for pavement design. According to IRC rate of growth of traffic is approximately 5-7.5% per annum.

(iii) Variation Chart: Variation charts shows the hourly, daily and seasonal variations. It helps in deciding the facilities and regulation needed during peak traffic hours.

(iv) Volume Flow Maps: Flow maps are prepared to show the distribution of volume by location. In volume flow maps, the width of the route is proportion to the volume for a specified period.

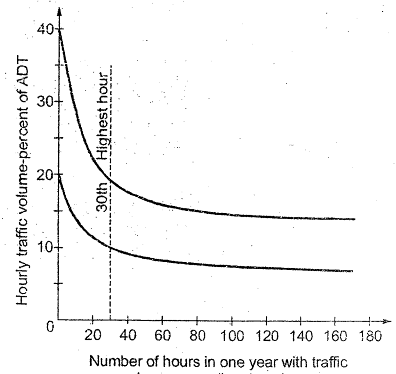


***Figure-4.1 :** Hourly traffic volumes*

At intersections, where the number of turning vehicles is important, flow maps may be prepared in a similar manner i.e. the width of band showing the volume.

(v) Thirtieths (30th) Highest Hourly Volume: It is that hourly volume which will be exceeded only 29 times in a year and all other hourly volume will be lesser than this as shown in Figure 4.1. It is used to design the highway elements and also called as design hourly volume.

Do You Know?

$$\text{Hourly Expansion Factor} = \frac{\text{Total traffic volume for a day}}{\text{Traffic volume for particular hour}}$$

$$\text{Daily Expansion Factor} = \frac{\text{Average total volume of a week}}{\text{Average volume for particular day}}$$

$$\text{Monthly Expansion factor} = \frac{\text{AADT}}{\text{ADT of particular month}}$$

Expansion factor value ≥ 1: It is equal to 1 only when traffic flows for a particular hour and there is no traffic flow during remaining hour. In this case total daily traffic is equal to hourly traffic.

Peak Hour Factor (PHF)

It is defined as the ratio between the number of vehicles counted during the peak hour and four times the number of vehicles counted during the highest 15 consecutive minutes. Peak hour factor is a measure of the variation in demand during the peak hour.

$$PHF_{15} = \frac{\text{Hourly traffic volume}}{\left(\frac{60}{15}\right) \times V_{15}(\max)}$$

$$PHF_{5} = \frac{\text{Hourly traffic volume}}{\left(\frac{60}{5}\right) \times V_{5}(\max)}$$

where, V_{15} = maximum number of vehicle during any 15 consecutive minutes
V_5 = maximum number of vehicle during any 5 consecutive minutes

NOTE: Value of Peak Hour Factor (PHF) varies between 0.25 and 1.00

4.3.2 Traffic Speed Studies

Speed is the rate of travel expressed in kmph or in m/s. Over a particular route, the actual speed of vehicle may vary. Speed of a vehicle depends upon several factors such as geometric features, traffic conditions, time, place, environment and driver.

Types of Speed

(i) Spot Speed: It is the instantaneous speed of a vehicle at any specified point. The spot speeds are affected by physical features of road such as pavement width, curve, sight distance, gradient, pavement unevenness and road side developments.

NOTE: Travel time is the reciprocal of speed and it is a measure of efficiency of roads.

There are a number of methods to measure spot speed:

(a) Enoscope Method or Mirror Box Method
(b) Photographic Method
(c) Electronic Method
(d) Radar Speed Meter Method

Most simplest method of finding spot speed is by using Enoscope.

Enoscope Method: In this method, an *L*- shaped box shown in Figure 4.2, open at both ends, with mirrors set inside at 45° angle is used. An observer is stationed on one side of road and starts a stopwatch when a vehicle crosses that section. An enoscope is placed at a convenient distance of say 50 m in such a way that the image of vehicle is seen by the observer when the vehicle crosses the section where enoscope is fixed. The greatest disadvantage is that the progress is so slow as it is difficult to spot out typical vehicles and the main advantage of enoscope method is that it is simple and cheep.

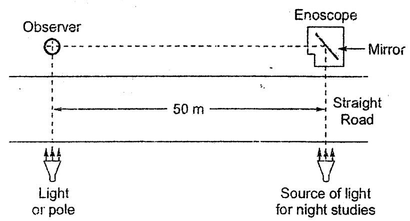


***Figure-4.2 :** Spot speed by enoscope*

Presentation of Spot Speed Data:

(a) Average Speed of Vehicle: From the spot speed data of the selected sample frequency distribution tables are prepared by arranging the data in groups of various speed ranges. The arithmetic mean is taken as average speed.

(b) Cumulative Speed of Vehicles: A graph is plotted between the cumulative frequency and speed with speed group on x-axis and vehicles travelled at or below the different speeds on the y-axis as shown in Figure 4.3. The 85th percentile speed is generally considered as the safe speed limit.

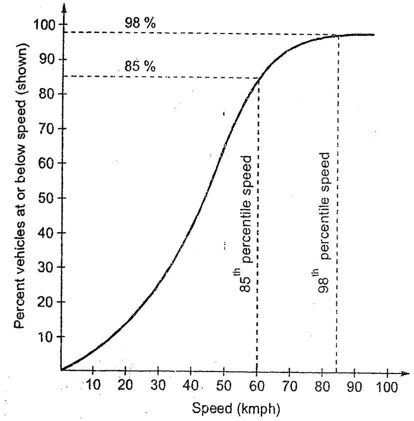


***Figure-4.3 :** Cumulative speed distribution*

NOTE: 85th percentile is that speed at or below 85 % of the vehicles are passing the point on the highway or only 15 % vehicles exceed the speed at that spot.

98th percentile speed is taken for the purpose of highway geometric design. 15th percentile speed is considered to be the lower safe speed limit to avoid congestion. 50th percentile speed is known as median speed at which 50% vehicles are moving above and 50% are moving below at that speed.

(c) Modal Average: Curve plotted between average values of each speed group on the x-axis and the frequency of vehicles in that group on y-axis is known as speed distribution curve as shown in Figure 4.4. Speed at which maximum number of vehicles are moving is termed as model speed.

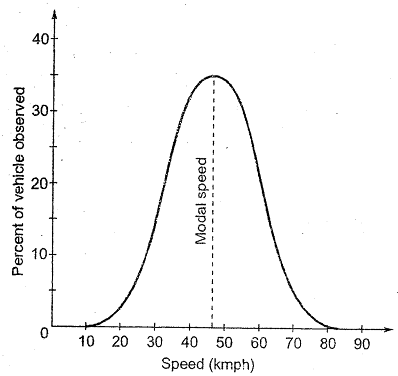


***Figure-4.4 :** Frequency distribution curve of the spot speeds*

Example-4.2 Spot speed studies were carried out at a certain stretch of a highway. The data collected by the spot speed studies carried out on a stretch highway is given below:

Speed range (kmph)	No. of vehicles observed	Speed range (kmph)	No. of vehicles observed
0 to 10	12	50 to 60	255
10 to 20	18	60 to 70	119
20 to 30	68	70 to 80	43
30 to 40	89	80 to 90	33
40 to 50	204	90 to 100	9

Determine the design speed for the design geometric elements of highway, modal average speed and speed limits for regulation of mixed traffic flow.

Solution:

Using the values of mid-speed and cumulative frequency percentage of below table.

Speed range (kmph)	Mid speed (kmph)	Frequency (f)	Frequency (%)	Cumulative frequency (%)
0 - 10	5	12	1.41	1.41
10 - 20	15	18	2.12	3.53
20 - 30	25	68	8.00	11.53
30 - 40	35	89	10.47	22.00
40 - 50	45	204	24.00	46.00
50 - 60	55	255	33.00	76.00
60 - 70	65	119	14.00	90.00
70 - 80	75	43	5.06	95.06
80 - 90	85	33	3.88	98.94
90 - 100	95	9	1.06	100.00
	Total	850	100.00	

(*i*) Design speed of highway = 98th percentile speed = 82.58 kmph

(*ii*) Lower speed limit for regulation = 15th percentile speed = 28.41 kmph

(*iii*) Upper speed limit for regulation = 85th percentile speed = 61.43 kmph

(*iv*) Modal average speed = 55 kmph

Do You Know?

- **Travel time:** It is an inverse of speed and it measure efficiency of road.

$$T = \frac{1}{V(\text{km/hr})} = \frac{3600}{V}\text{sec}$$

- **Spot speed:** It is an instantaneous speed of any vehicle at any instant of time.

(ii) **Average Speed:** It is the average of spot speed of vehicles at a particular section or location. There are two types of average speed or mean speed:

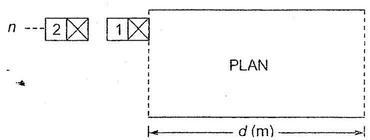
(a) Time Mean Speed: It is defined as the average speed of all the vehicles passing a point on a highway over some specified time interval.

Time mean speed, $$V_t = \frac{\sum_{i=1}^{n} V_i}{n}$$

where, V_i = Instantaneous speed of i^{th} vehicles in kmph

n = Number of vehicles observed

(b) Space Mean Speed: It is the average speed of all vehicle at a certain road length over some specified time period. It is obtained from the observed travel time of the vehicles over a relatively long stretch of the road. Space mean speed is the harmonic mean of the speed of the vehicles passing a point on a highway during a particular interval of time.



Space mean speed, $$V_s = \frac{3.6\,dn}{\sum_{i=1}^{n} t_i} = 3.6\left[\frac{n}{\frac{t_1}{d_1}+\frac{t_2}{d_2}+\ldots\frac{t_n}{d_n}}\right] = 3.6\left[\frac{n}{\frac{1}{V_1}+\frac{1}{V_2}+\ldots\frac{1}{V_n}}\right]$$

where, d = Length of road considered in m

n = Number of individual vehicle observations

t_i = Observed travel time (in sec) for i^{th} vehicle to travel distance d

Space mean speed is harmonic mean of all the speed.

Remember

If the speed study data is given in the form of frequency table, then

Time mean speed, $$v_t = \frac{\sum q_i v_i}{\sum q_i}$$

where, q_i = Number of vehicles in i^{th} speed range

Space mean speed, $$v_s = \frac{\sum q_i}{\sum \frac{q_i}{v_i}}$$

(iii) Running Speed: It is the average speed maintained by a vehicle over a particular stretch of road, while the vehicle is in motion.

$$\text{Running speed} = \frac{\text{Total distance travelled}}{\text{Total running time}}$$

(iv) Travel Speed or Journey Speed: It is defined as the total distance travelled upon total time taken including all stoppage and delay.

Do You Know?

- Time mean speed is greater than space mean speed. As arithmetic mean is greater than the harmonic mean.
- When vehicles are moving with same speed then space mean speed is equal to time mean speed.

Interrelationship between Space mean speed and Time mean speed

$$V_t = V_s + \frac{\sigma_s^2}{V_s} \quad \text{and} \quad V_s = V_t - \frac{\sigma_t^2}{V_t}$$

where σ_s^2 and σ_t^2 are variance for space mean speed and time mean speed respectively.

Type of Speed Studies

Generally there are two types of speed studies:

(i) Spot Speed Study: Spot speed studies cannot be used to find density because measurements are done at one point only. Spot speed studies are conducted mainly for the following purposes:

(a) Design of traffic signals
(b) Analysis of high accident location
(c) Used for geometric design
(d) Determining the speed trends

(ii) Speed and Delay Study: This study is made mainly for the following purposes :

(a) It gives the running speeds, overall speeds, fluctuation in speeds and delay between two stations.
(b) To find the density of traffic.
(c) Economic studies utilise travel time and delay data.
(d) It also gives the information of amount, location, duration frequency and cause of delay in the traffic.

Delay can be categorized into two types:

(a) **Fixed delay:** It is the delay to which traffic is subjected to regardless of the amount of traffic volumes and interferences present on the highway. This is not due to the characteristics of traffic streams. This includes traffic signals, stop signals, railroad crossings, etc. This delay can occur even with only one vehicle on the highway.

(b) **Operational delay:** This is also known as congestion delay. This is the delay caused by interference with other components of traffic. The difference between travel time over a route during an extremely low and during very high traffic volume indicates the amount of operational delay.

There are various methods of carrying out speed and delay study:

(a) License plate or vehicle number method
(b) Interview technique
(c) Elevated observation
(d) Photographic technique
(e) Floating car method or riding check method

License Plate Method

In this method observers are stationed at the entrance and exit of a test section where information of travel time is required. This method does not give important details such as causes of delays and the duration and number of delays within the test stretch.

Interviewed Technique

In this method interview of road users on the spot is used to collect the details.

Elevated Observations and Photographic Technique

This method is useful for studying short test sections like intersections.

Floating Car Method

In floating car method a test vehicle is driven over a given course of travel, approximately at the average speed of stream. A number of test runs are made along the study stretch and a group of observer records the various readings.

One observer with two stopwatches, notes time at various control points such as intersections and any other fixed points in each trip with one stopwatch and also notes the time of delay at each point with the help of second stopwatch.

Second observer notes the time, location and cause of the delays.

Third observer notes the number of vehicles overtaking the test vehicle and overtaken by the test vehicle.

Fourth observer notes the number of vehicles travelling in opposite direction in each trip.

The average journey time ($\bar{t}$) for all the vehicles in a traffic stream in the direction of flow is given as

$$\bar{t} = t_w - \frac{n_y}{q}$$

$$\text{Flow of vehicles, } q\ (\text{vehicle / min}) = \frac{n_a + n_y}{t_a + t_w}$$

where, n_a = Average number of vehicles counted in the direction of stream when the test vehicle travels in the opposite direction

n_y = The average number of vehicles overtaking the test vehicle – the number of vehicles overtaken when the test is in the direction of flow

q = Flow of vehicles in one direction of the stream

t_a = Average journey time when test vehicle is running against the stream q

t_w = Average journey time when the test vehicle is travelling with the stream q

Example-4.3 The data collected from speed and delay studies by floating car method, running North-South, on a stretch of 3.5 km are given below. Find out the average journey speed, running speed and volume of traffic speed along both the directions.

Direction of trip	Journey time (min-sec)	Total delay (min-sec)	No. of vehicles overtaking	No. of vehicles overtaken	No. of vehicles from opp. direction
N - S	6 - 32	1 - 40	4	7	268
S - N	7 - 14	1 - 50	5	3	186
N - S	6 - 50	1 - 30	5	3	280
S - N	7 - 40	2 - 00	2	1	200
N - S	6 - 10	1 - 10	3	5	250
S - N	7 - 00	2 - 22	2	2	170
N - S	6 - 25	1 - 40	2	5	290
S - N	7 - 30	1 - 40	3	2	160

Solution:

(i) North-South Direction: Table given below shows the mean values of journey time, stopped delay, number of vehicles overtaking, overtaken and in opposite direction for North-South

Direction	Journey time (min-sec)	Stopped delay (min-sec)	No. of Vehicles		
			Overtaken	Overtaking	In opposite direction
N - S	6 - 32	1 - 40	4	7	268
	6 - 50	1 - 30	5	3	280
	6 - 10	1 - 10	3	5	250
	6 - 25	1 - 40	2	5	290
Total	26 - 00	6 - 00	14	20	1088
Mean	6 - 30	1 - 32	3.5	5.0	272

$\Rightarrow \quad q = \text{Average volume} = \dfrac{n_a + n_y}{t_a + t_w}$

where, n_y = Average no. of vehicles overtaking minus overtaken = 3.5 – 5.0 = –1.5

n_a = Average no. of vehicles during trips in opposite direction (from $S-N$ trips) = 179

t_w = Average journey time = 6 min 30 sec = 6.5 min

t_a = Average journey time during trips against the stream = 7 min 36 sec= 7.6 min

$\Rightarrow \quad q = \dfrac{179-1.5}{7.6+6.5} = 12.59$ veh/min

$\therefore \quad \bar{t} = \text{Average journey time} = t_w - \dfrac{n_y}{q} = 6.5 - \dfrac{(-1.5)}{12.59} = 6.62$ min

Average journey speed $= \dfrac{3.5}{6.62}$ km/min $= \dfrac{3.5\times 60}{6.62} = 31.7$ kmph

Average stopped delay = 1.5 min

Average running time = Average journey time – Average stopped delay

= 6.62 – 1.50 = 5.12 min

Average running speed $= \dfrac{3.5\times 60}{5.12} = 41.0$ kmph

(ii) South-North Direction: Table given below shows the mean values of journey time, stopped delay, number of vehicles overtaking, overtaken and in opposite direction for South-North

Direction	Journey time (min-sec)	Stopped delay (min-sec)	No. of Vehicles		
			Overtaken	Overtaking	In opposite direction
S - N	7 - 14	1 - 50	5	3	186
	7 - 40	2 - 00	2	1	200
	7 - 00	2 - 22	2	2	170
	7 - 30	1 - 40	3	2	160
Total	30 - 24	7 - 12	12	8	716
Mean	7 - 36	1 - 40	3.0	2.0	179

where, n_y = 3.0 – 2.0 = 1.0

n_a = (from $N-S$ strips) = 272

t_w = 7.6 min and t_a = 6.5 min

$\Rightarrow \quad q = \dfrac{272+1.0}{6.5+7.6} = 19.36$ veh/min

$\therefore \quad \bar{t} = 7.6 - \dfrac{1.0}{19.36} = 7.55$ min

Journey speed $= \dfrac{3.5\times 60}{7.55} = 28.8$ kmph

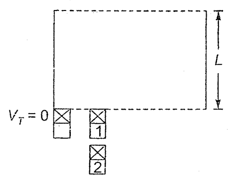
Average stopped delay = 1.8 min

Average running time = 7.55 – 1.80 = 5.75 min

Average running speed $= \dfrac{3.5\times 60}{5.75} = 36.5$ kmph

Moving Vehicle Estimation Method

Case-I: When test vehicle is stopped and all other vehicles are moving.

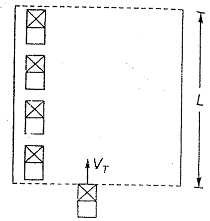


Traffic flow, $q = \dfrac{n_0}{T}$

where T = Duration of study

n_0 = Number of Vehicle overtaking the test vehicle

Case-II: When test vehicle is moving and all other are stopped.



Traffic density, $K = \frac{n_s}{L}$

$\Rightarrow$ $n_s = KL = KV_T T \qquad (\because L = V_T T)$

where n_s = number of vehicle stopped or overtaken by the test vehicle

$$n = n_0 - n_s = qT - KV_T T$$

$$\frac{n}{T} = q - KV_T$$

when test vehicle moves in direction of stream (S – N)

$$n = n_y \text{ and } T = T_w$$

$$\Rightarrow \qquad \frac{n_y}{T_w} = q - K\frac{L}{T_w} \qquad ...(i)$$

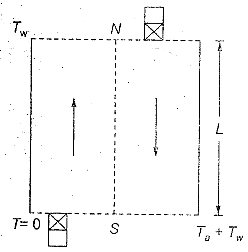
when test vehicle moves against the stream (N – S)

$$\Rightarrow \qquad \frac{n_a}{T_a} = q + \frac{KL}{T_a} \qquad ...(ii)$$

$$n = n_0 - n_s$$

$$n = n_a - n_s$$

$$n = n_a \qquad (\because n_s = 0)$$



From eq. (i)

$$n_y = qT_w - KL \qquad ...(iii)$$

From eq. (ii)

$$n_a = qT_a + KL \qquad ...(iv)$$

Adding eq. (iii) and (iv)

$$n_a + n_y = q(T_a + T_w)$$

$$q = \frac{n_a + n_y}{T_a + T_w}$$

From eq. (i)

$$\frac{n_y}{T_w} = q - \frac{KL}{T_w}$$

$$q_{(veh/hr)} = K_{(veh/km)} \times V_{(km/hr)}$$

$$\frac{n_y}{T_w} = q - \frac{q}{V} \times \frac{L}{T_w}$$

$$\frac{n_y}{T_w} = q - q \times \frac{\bar{t}}{T_w}$$

$$\frac{n_y}{T_w} = q\frac{(T_w - \bar{t})}{T_w}$$

$$\bar{t} = T_w - \frac{n_y}{q}$$

4.3.3 Origin and Destination Studies

This study is generally carried out to

(*i*) Plan the road network and other facilities for vehicular traffics.

(*ii*) Plan the schedule of different modes of transportation for the trip demand.

(*iii*) To judge the adequacy of existing routes and to use in planning new network to roads.

(*iv*) To locate expressway or major routes along the desires lines.

(*v*) To establish preferential routes for various categories to vehicle including by pass.

(*vi*) To locate intermediate stops of public transport.

(*vii*) It is also used for mass rapid transit system.

There are number of methods for collecting the *O* and *D* data:

(i) Road Side Interview Method: Data is collected quickly in short duration and the field organisation is simple and the team can trained quickly. The main disadvantage of this method is that the vehicles are stopped for interview and there is delay to the vehicular movement.

(ii) License Plate Method: This method is quite easy and quick as for as the fieldwork is concerned. Observers are simultaneously stationed at all points of entry and exit leading into and out of a particular area. After collecting field data computation and analysis, by tracking each vehicle number and its time of entering and leaving the considered area. This method is advantageous, if the area considered is small.

(iii) Return Post Car Method: In this method post cards with return address are distributed to the road users. Distributing stations for the post cards may be selected where vehicles have to stop such as toll booth.

Questionnaire to be filled in by the road user is printed on the card, along with a request for cooperation and purpose of study. This method is suitable where the traffic is heavy.

(iv) Tag on Car Method: A precoded card is stuck on the vehicle as it enters the area under study. When the car leaves the study area then the other observations are recorded on the tag. This method is suitable when traffic is heavy and move continuously.

(v) Home Interview Method: Random people are selected from the marked area and visited by a expert term who collect all the travel data from each member of the household. The problem of stopping vehicle and consequent difficulties are avoided altogether. Additional data including socio-economic and other details may be collected so as to be useful for force casting traffic and transportation growth.

(vi) Work Spot Interview Method: The transportation needs of work trips can be planned by collecting the *O* and *D* data at work spots like offices, factories, educational institutions etc. by personal interviews.

Presentation of *O* and *D* Data

The *O* and *D* data are presented in the following form:

(i) *O* and *D* Table: These are prepared showing number of trips between different zones.

(ii) Desire Lines: These are the straight lines connecting the origin points with destinations. Desire lines density map shows the actual desire of road user based on which necessity of new road link or bypass is decided. The width of desire line is proportional to the number of trips in both directions.

(iii) Pie Charts: Diameter of circles are proportional to number of trips.

(iv) Contour Lines: The shape of the contours would indicate the general traffic need of the area.

4.3.4 Traffic Flow Characteristics and Studies

Traffic flow characteristics are divided under two categories:

1. **Macroscopic characteristics:** Traffic flow theory assumes that there is a fundamental relationship among the three principle variables of traffic flow, speed, and density as follows:

$$q = k \times U \quad \text{...(i)}$$

where, q = Traffic volume (Vehicles / hour)
k = Traffic Density (Vehicles /km)
U = Speed of vehicle (kmph) i.e. space mean speed

Eq. (i) can be derived to get the relationship between any two variables in the three principal variables (e.g., speed-density), and the other two relationships (density-flow, speed-flow) will be got automatically. Therefore, we choose the speed density k as a function of speed U, then the speed-density function can be shown as $k = k(U)$.

The first traffic flow model was proposed by Greenshields in 1935. He suggested a linear relationship between the density and speed

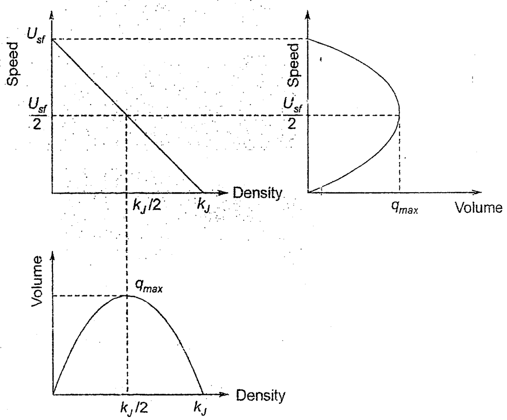
$$U = U_{sf}\left(1 - \frac{k}{k_J}\right)$$

where, k_J = jam density (corresponding to zero speed, i.e., $U = 0$); U_{sf} = free flow speed.

and Traffic volume,

$$q = kU$$

$$\Rightarrow \quad = U_{sf}\left(k - \frac{k^2}{k_J}\right)$$



For maximum volume,

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

$$\Rightarrow \quad U_{sf}\left(1 - \frac{2k}{k_J}\right) = 0$$

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

when, $k = \frac{k_J}{2}$ and $U = \frac{U_{sf}}{2}$

$$\therefore \quad q_{max} = \frac{U_{sf}.k_J}{4}$$

A number of mathematical models between the traffic speed and density were proposed and calibrated by fitting curves to empirical traffic data.

Green berg model: $U = U_{sf} \ln\frac{k_J}{k}$

Underwood model (exponential distribution) : $U = U_{sf}e^{-k/k_J}$

2. **Microscopic characteristics:**

Time Headway: The time interval between the passage of successive vehicles moving in the same lane and measured from head to head as they pass a point on the road is known as the time headway.

Space Headway: The distance between successive vehicles moving in the same lane measured from head at any instance is the space headway.

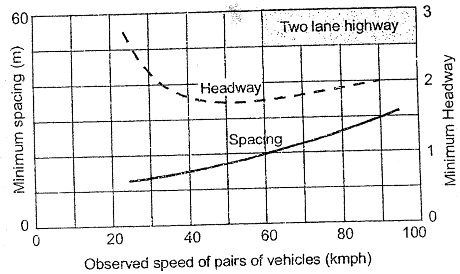


***Figure-4.5**: Variation of min. space and time headways with speed*

Space gap allowed by the driver of a followed vehicle depends on several factors such as:

(i) Speeds of leading and following vehicles
(ii) Type and characteristics of the two vehicles
(iii) Driver characteristics of the following vehicle
(iv) Level of service
(v) Road geometrics
(vi) Environmental factors

Traffic stream generally has flow and counter flow along a common route. The basic traffic manoeuvres are diverging, merging and crossing as shown in **Figure 4.6**.

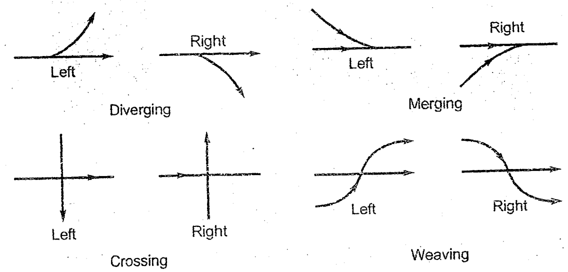


***Figure-4.6**: Traffic manoeuvres*

When a vehicle move obliquely across the path of another vehicle moving in the same direction, as relatively small angle of crossing, the action is termed is weaving. The weaving manoeuvre may also consist of merging and diverging operations.

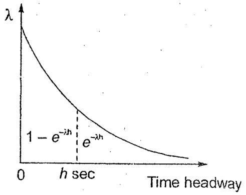
Remember

- The number of headways per unit time is dependent on the rate of traffic flow and is therefore a direct measure of traffic volume.
- With increase in speed of traffic stream, the minimum space headway increases whereas the minimum time headway first decreases and after reaching a minimum value at optimum speed on the stream increases.
- Maximum flow or capacity flow is attained at this speed when the time headway is minimum.

Vehicle Arrival Distribution Rate

Probability $f(t) = \lambda e^{-\lambda t}$ (exponential distribution)

Probability that time headway lies between 0 and ∞.



$$P[0 \le t \le \infty] = \int_0^{\infty} \lambda e^{-\lambda t} dt = \lambda \left[\frac{e^{-\lambda t}}{-\lambda}\right]_0^{\infty}$$

$$= \lambda\left[0 - \frac{1}{-\lambda}\right] = 1$$

$$P[h \le t \le \infty] = \int_h^{\infty} \lambda e^{-\lambda t} dt = \lambda \left[\frac{e^{-\lambda t}}{-\lambda}\right]_h^{\infty} = e^{-\lambda h}$$

Poisson Distribution of Vehicle Arrivals

The distribution of vehicles in space or in time may assume various mathematical forms. In general, it is likely that the spacing between vehicles will be distributed in a random manner with gaps of various sizes. One of the statistical models to describe the distribution is the Poisson distribution.

As n becomes large, the binominal distribution approaches the Poisson distribution.

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

where, $P(n)$ = Probability of arrival of n vehicles in any internal of t sec

λ = average rate of arrival (vehicle per unit time)

t = time internal

e = base of natural logarithms

Example-4.4 It was observed that 150 vehicle crossed a particular location of highway in 30 minutes. Assume that vehicle arrival follow a negative exponential distribution. Find the number of time headways greater than 5 sec in above observation.

Solution:

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

Where, $P(x)$ is the probability of x events (vehicle arrivals) in some time interval (t).

λ is the mean arrival rate in that interval,

$$\lambda = \frac{150}{30 \times 60} \times 5 = \frac{5}{12}$$

Now, the probability that zero vehicle arrive in an interval t, denoted as P(0), will be same as the probability that the *headway* (into arrival time) *greater than or equal to t*.

$$\therefore \quad P(x=0) = \frac{\lambda^0 e^{-\lambda}}{0!} = e^{-5/12} = 0.6592$$

$$\therefore \quad P(x=0) = 0.6592$$

Number of time headway in 30 minutes = 0.6592 × 30 × 60 = 1186.63

4.3.5 Traffic Capacity Studies

Some important related terms which are often used are:

(i) Traffic Volume (q): It is the number of vehicles moving in a specified direction on a given lane or roadway that pass a given point during specified unit of time. It is expressed as vehicles per hour or vehicles per day.

(ii) Traffic Density (k): It is defined as the number of vehicles occupying a unit length of lane of roadway at a given instant. It is expressed in vehicles per kilometre.

NOTE The highest traffic density will occur when the vehicles are practically at a stand still on a given route and in this case traffic volume will approach zero. Volume represents an actual rate of flow and responds to variations in traffic demand, while capacity indicates a capability or maximum rate of flow with a certain level of service characteristics that can be carried by the roadways.

(iii) Traffic Capacity: It is the ability of a roadway to accommodate traffic volume. It is expressed as vehicles per hour per lane. The capacity of roadway depends on a number of prevailing roadway and traffic conditions. Traffic capacity is always greater than or equal to traffic volume.

(iv) Basic Capacity: It is the maximum number of vehicles that can pass a given point on a lane or roadway during one hour under the most nearly ideal roadway and traffic conditions which can possibly be at trained.

(v) Possible Capacity: It is the maximum number of vehicles that can pass a given point on a lane or roadway during one hour under prevailing roadway and traffic conditions. The value of possible capacity varies between zero to basic capacity.

(vi) Practical Capacity: It is the maximum number of vehicle that can pass a given point on a lane or roadway during one hour, without traffic density being so great as to cause unreasonable delay, hazard or restriction to the drivers freedom to manoeuvre under the prevailing roadway and traffic conditions. This is also known as design capacity.

For design purpose we neither use basic capacity nor possible capacity. Practical capacity is used as design capacity.

Example-4.5 On an urban road, the free mean speed was measured as 70 kmph and the average spacing between the vehicles under jam condition as 7.0 m. The speed-flow-density equation is given by:

$$U = U_{sf}\left[1 - \frac{k}{k_j}\right] \text{ and } q = U \times k$$

where,

U = space-mean speed (kmph) $\quad U_{sf}$ = free mean speed (kmph)

k = density (veh/km) $\quad k_j$ = jam density (veh/km)

q = flow (veh/hr)

The maximum flow (veh/hr) per lane for this condition will be equal to

(a) 2000 (b) 2500

(c) 3000 (d) None of the above

Ans. (b)

We know that,

$$\text{Traffic volume} = \text{Density} \times \text{Speed}$$

$$q = k \times U$$

$$\Rightarrow \quad q = U_{sf}\left(1 - \frac{k}{k_j}\right)k = U_{sf}\left(k - \frac{k^2}{k_j}\right)$$

Now, Jam density, $k_j = \dfrac{1000}{\text{average spacing between vehicles}} = \dfrac{1000}{7}$

Also, for maximum traffic volume, $\dfrac{dq}{dk} = 0$

$\Rightarrow \qquad U_{sf}\left(1-\dfrac{2k}{k_j}\right) = 0 \qquad$ But $U_{sf} \neq 0$

$\Rightarrow \qquad 1-\dfrac{2k}{k_j} = 0 \Rightarrow k = \dfrac{k_j}{2}$

$\therefore$ Maximum traffic volume,

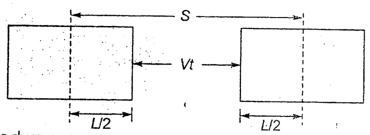
$$q_{max} = U_{sf}\left(\frac{k_j}{2} - \frac{(k_j/2)^2}{k_j}\right) = U_{sf}\left(\frac{k_j}{2} - \frac{k_j}{4}\right) = U_{sf}\left(\frac{k_j}{4}\right) = 70 \times \frac{1000}{7\times 4} = 2500 \text{ veh/hr}$$

Determination of Theoretical Maximum Capacity

(*i*) Maximum theoretical capacity on the basis of space headway:

Theoretical maximum capacity of a lane, $C = \dfrac{1000\,V}{S}$

where, C = Capacity of single lane (vehicle per hour)
V = Speed of vehicle (kmph)
S = Average centre to centre spacing of vehicles = $S_g + L$, or
= $0.7v_B + L$, where v_B in m/s
S_g = Minimum space gap = 0.278 Vt
L = average length of vehicle
t = Reaction time = 0.7 sec



(*ii*) Maximum theoretical capacity on the basis of time headway:

Theoretical maximum capacity of a lane, $C = \dfrac{3600}{H_t}$

where, H_t = Minimum time headway in seconds

Relationship between Speed and Maximum Capacity of a Traffic Lane

The peak value of the theoretical maximum capacity is reached at an optimum speed. As the further increase in speed will decreases the maximum capacity of the lane.

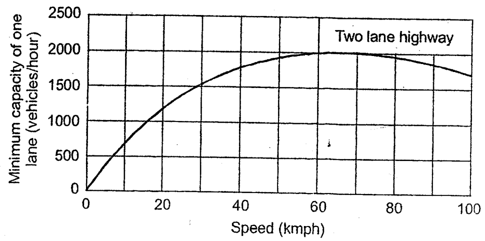


***Figure-4.7 :** Speed and Capacity relation*

Example-4.6 Determine the theoretical capacity of a traffic lane having one way traffic flow at a stream speed of 40 kmph. Assume the average space gap between vehicles to follow the relation S_g = 0.278 Vt where 'V' is the stream speed in kmph, 't' is the average reaction times = 0.7 sec and assume average length of vehicles = 5.0 m.

Solution:

V = 40 kmph, t = 0.7 sec, L = 5.0 m

$S = 0.278\,Vt + L = 0.278 \times 40 \times 0.7 + 5.0 = 12.78$ m

Theoretical capacity, $C = \dfrac{1000\,V}{S} = \dfrac{1000 \times 40}{12.78} \simeq 3130$ vehicles/hour/lane

Factors Affecting Practical Capacity

There are various factors which affects the practical capacity are:

(i) Lane width: As lane width decreases, the capacity also decreases.

(ii) Lateral clearance: Restricted lateral clearance affects driving comfort, increases accident rates and reduces capacity.

(iii) Width of shoulder: Narrow shoulder reduces the effective width of traffic lanes, thus reduce capacity of lane.

(iv) Commercial vehicle: Large commercial vehicles occupy more space, may travel at slow speeds and influence the traffic in the same lane as well as the adjoining lanes.

(v) Alignment: Restrictions to sight distance requirements cause reduction in capacity.

(vi) Presence of intersection at grade: Intersection restricts the free flow of traffic and thus adversely effect the capacity.

(vii) Other factors are stream speed one or two way movement of traffic, number of lanes and traffic volume.

The practical capacity values suggested by IRC for the purpose of design of different types of roads in rural areas and urban roads are shown in Table 4.1.

Table-4.1 : Capacity of different types of roads

Type of Road	Capacity (PCU per day in both direction)
Single lane roads having a 3.75 m wide carriageway with normal earthen shoulders.	1000
Single lane roads having a 3.75 m wide carriageway with adequately designed shoulders 1.0 m wide.	2500
Two lane roads having a 7 m wide carriageway with normal earthen shoulders.	10000
Roads of intermediate width i.e., having a carriageway of 5.5 m with normal earthen shoulders.	5000

Design Capacity And Level of Service

When all the vehicles flow as a stream at a optimum speed with no opportunity of overtaking at that time flow is called as capacity flow. At this optimum speed, volume to capacity ratio approaches a maximum possible value of 1.0.

Six level of services *A, B, C, D, E* and *F* are recommended by Highway Capacity Manual as shown in Figure 4.8.

(i) Level of service A exist when volume to capacity is so low that users has freedom to select desired speeds and manoeuvre within the traffic stream. Level of comfort and convenience to users is extreme.

(ii) With increase in the volume to capacity ratio, the operating speeds and overtaking opportunities reduces and level of service fall to decreasing values of B, C, D and E.

(iii) In the level of service F, flow and speed of vehicle reduces which leads to congestion. This is the lowest level of service.

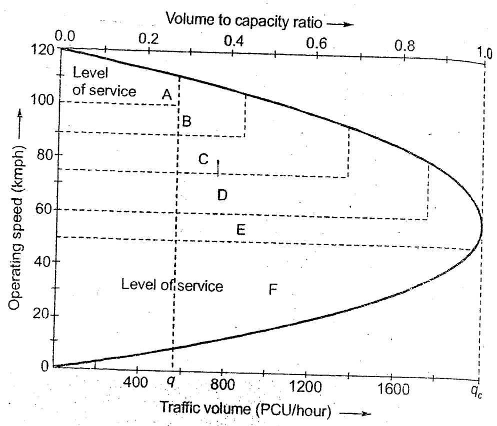


***Figure-4.8**: General concept of level of service*

4.3.6 Parking Studies

Parking is one of the major problem that is created by the increasing road traffic, especially in metropolitan cities. The availability of less space in urban areas has increased the demand for parking space especially in areas like central business district.

Various aspects to be investigated during parking studies are as follows:

(i) Parking Demand: Methods to measure parking demand is by making cordon counts recording accumulation of vehicles during the peak hours by subtracting the outgoing traffic from the traffic volume entering the cordoned area.

Do You Know?

- Interview technique is useful when parking demand is high.
- Another method is by counting the number of vehicles parked in the area under study during different periods of the day. This method is useful when the parking demand is less than the space available for parking.

(ii) Parking Characteristic: The study is directed to note the present parking practices prevalent in the area under study and the problems in parking.

(iii) Parking Space Inventory: The area under study is fully surveyed and a map is prepared showing all places where kerb parking and offstreet parking facilities can be provided to meet the parking demands.

Parking Statistics

(i) Parking Accumulation: It is defined as the number of vehicles parked at a given instant of time. Normally this is expressed by accumulation curve. Accumulation curves is the graph obtained by plotting the number of bays occupied with respect to time.

(ii) Parking Volume: Parking volume is the total number of vehicles parked at a given duration of time. This does not account for repetition of vehicles.

(iii) Parking Load: Parking load gives the area under the accumulation curve. It can also be obtained by simply multiplying the number of vehicles occupying the parking area at each time interval with the time interval. It is expressed as vehicle hours.

(iv) Average Parking Duration: It is the ratio of total vehicle hours to the number of vehicles parked

$$\text{Parking Duration} = \frac{\text{Parking Load}}{\text{Parking Volume}}$$

(v) Parking Turnover: It is the ratio of number of vehicles parked in a duration to the number of parking bays available.

$$\text{Parking Turnover} = \frac{\text{Parking Load}}{\text{No. of bays available}}$$

This can be expressed as number of vehicles per day per time duration.

(vi) Parking Index: It is also called occupancy or efficiency. It is defined as the ratio of number of bays occupied in a time duration to the total space available. Parking index can be found out as follows

$$\text{Parking Index} = \frac{\text{Parking Load}}{\text{Parking Capacity}} \times 100$$

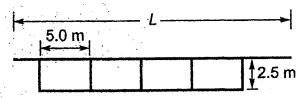
Kerb Parking / On Street Parking

In this type of parking vehicles are parked on the kerb which is designed for parking. Angle parking accommodates more vehicle per unit length but maximum vehicles can be parked with an angle of 90°.

As per IRC the standard dimensions of a car is taken as 5 × 2.5 m and that for a truck is 3.75 × 7.5 m. Generally used angle of parking are 30°, 45°, 60°, 90°.

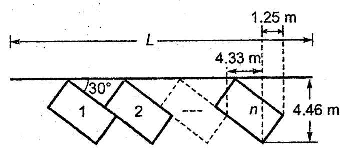
For parallel parking:

Number of spaces, $N = \frac{L}{6.6}$

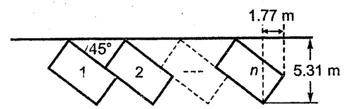


(Where, L = Length of parking)

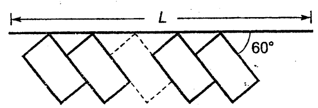
For 30° parking: $N = \frac{L-0.85}{5.1}$



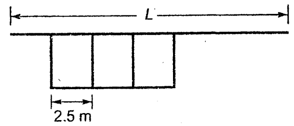
For 45° parking: $N = \frac{L-2}{3.6}$



For 60° parking: $N = \frac{L-2}{2.9}$



For 90° parking: $N = \frac{L}{2.5}$



Offstreet Parking

It is a type of parking provided at a separate place away from kerb as shown in **Figure 4.9**. The main advantage of this type of parking is that there is no disturbance on the road during parking.

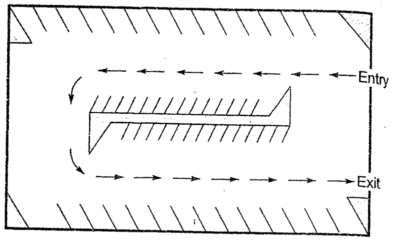


***Figure-4.9 :** Offstreet parking*

4.3.7 Accident Studies

One of the main objectives of traffic engineering is to provide safe traffic movements. Road accidents cannot be prevented but they can be reduced considerably by suitable traffic engineering. So, it is essential to analyse every individual accident and maintain zonewise accident records.

Objectives of the Accident Studies:

(*i*) To study the cause of accident and suggest corrective treatment

(*ii*) To support proposed designs

(*iii*) To make computation of financial losses

(*iv*) To demonstrate the improvement in the problem

(*v*) To give economic justification for the suggested improvements

Causes of Accidents:

Basically four factors are involved in road accidents :

(*i*) The vehicle — (*ii*) The road and its condition

(*iii*) The road user — (*iv*) The environment

Mathematical Analysis of Accident Studies

The following assumptions involved in the analysis of accidents are:

(*i*) If skid marks are present, then it is assumed that 100% skid occurs.

(*ii*) If skid marks are not present then free collision is assumed means no brakes are applied.

According to Newton's law of collision,

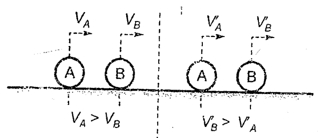
Coefficient of restitution, $$e = \frac{\text{Velocity of separation}}{\text{Velocity of approach}}$$

Value of coefficient of restitution lies between 0 and 1.

$e = 0$, for perfectly plastic collision

$e = 1$, for perfectly elastic collision

$$e = \frac{V_{B'} - V_{A'}}{V_A - V_B}$$



For perfectly elastic collision, $e = 1$

$$V_A - V_B = V_{B'} - V_{A'}$$

For perfectly plastic collision, $e = 0$

$V_{B'} = V_{A'}$, which means both will move together

Different Cases of Accidents

Case-1: When moving vehicle hits the parked vehicle.

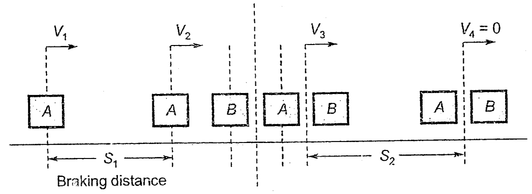


***Figure-4.10 :** Moving vehicle A hits parked vehicle B*

Let,
s_1 = skid mark length between v_1 and v_2
s_2 = skid mark length after collision to stop condition
f = coefficient of friction

(*i*) Energy conservation equation just before the collision.

$$\frac{1}{2}mv_1^2 - \frac{1}{2}mv_2^2 = fws_1 = fmgs_1$$

$$v_1^2 - v_2^2 = 2gfs_1 \quad \ldots(i)$$

(*ii*) Conservation of momentum just before and after the collision

$$m_Av_2 + m_B \times 0 = (m_A + m_B)\, v_3$$

$$m_Av_2 = (m_A + m_B)\, v_3 \quad \ldots(ii)$$

(*iii*) Conservation of energy equation after the collision

$$\frac{1}{2}mv_3^2 - \frac{1}{2}m(0)^2 = fmgs_2$$

$$v_3^2 = 2gfs_2 \quad \ldots(iii)$$

where, m_A and m_B are masses of vehicle *A* and *B*.

Example-4.7 A vehicle weighing 2.0 kN skids through a distance equal to 40 m before colliding with another parked vehicle of weight 1.0 kN. After collision both the vehicles skid through a distance equal to 12 m before stopping. Determine the initial speed of the moving vehicle. Assume coefficient of friction as 0.5.

Solution:

Let, the original speed of the vehicle be v_1 m/s reduced to v_2 m/s by applying brakes and skidding distance s_1 = 40 m.

Now just after the collision both vehicles *A* and *B* start moving together with speed v_3 m/s and finally stop, v_4 = 0, after skidding through a distance s_2 = 12 m.

(a) After collision:

Loss in kinetic energy of both vehicles together = Work done against frictional force

i.e., $$\frac{(W_A + W_B)}{2g}(v_3^2 - v_4^2) = (W_A + W_B)\, f s_2$$

$$\Rightarrow \quad \frac{v_3^2}{2g} = 0.5 \times 12$$

$$\therefore \quad v_3 = \sqrt{2 \times 9.8 \times 0.5 \times 12} = 10.85 \text{ m/s}$$

(b) At collision:

Momentum before impact = Momentum after impact

i.e.,
$$\frac{W_A v_2}{g} = \frac{(W_A + W_B)v_3}{g}$$

$$\therefore \quad v_2 = \frac{(W_A + W_B)}{W_A} v_3 = \frac{3}{2} \times 10.85 = 16.27 \text{ m/s}$$

(c) Before collision:

Loss in kinetic energy = Work done against braking force in reducing the speed

$$\frac{W_A}{2g}(v_1^2 - v_2^2) = W_A \times f \times s_1$$

$$v_1^2 = 2g f s_1 + v_2^2 = 2 \times 9.81 \times 0.5 \times 40 + 16.27^2$$

$$v_1 = 25.63 \text{ m/s}$$

Hence the initial speed of moving vehicle,

$$V_1 = 3.6 \times 25.63 = 92.27 \text{ kmph}$$

Case-2: When two vehicles moving towards crossing collides at intersection.

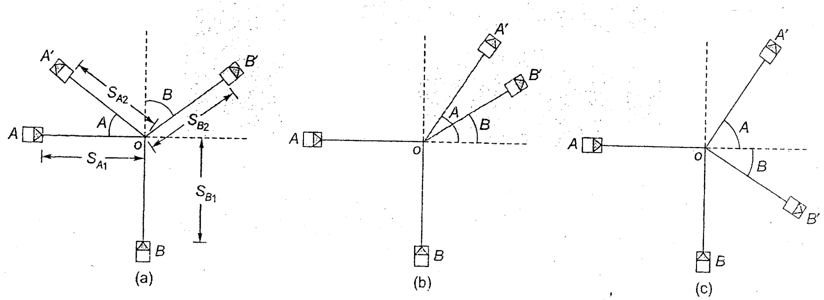


***Figure-4.11 :** Collision of two vehicle approaching from right angles*

(*i*) Energy conservation before collision:

$$\frac{1}{2}mv_{A_1}^2 - \frac{1}{2}mv_{A_2}^2 = fmgs_{A1}$$

$$v_{A_1}^2 - v_{A_2}^2 = 2g\, fs_{A1}$$

Similarly,
$$v_{B_1}^2 - v_{B_2}^2 = 2\, g\, fs_{B1}$$

(*ii*) Conservation of momentum at the time of collision:

Along $W-E$: $\quad m_A v_{A_2} + m_B \times 0 = m_A v_{A_3} \cos\theta_A + m_B v_{B_3} \sin\theta_B$

Along $S-N$: $\quad m_A \times 0 + m_B v_{B_2} = m_A v_{A_3} \sin\theta_A + m_B v_{B_3} \cos\theta_B$

(*iii*) Energy conservation after collision:

$$\frac{1}{2}mv_{A3}^2 - 0 = fmgs_{A2}$$

$$v_{A3}^2 = 2g\, fs_{A2}$$

Similarly,
$$v_{B3}^2 = 2g\, fs_{B2}$$

Example-4.8 **Two vehicles *A* and *B* weighing 4.4 tonnes and 6.0 tonnes respectively, *A* from West and *B* from South, collide with each other at a right angle intersection. After the collision, vehicle *A* skids distance of 15 m in a direction 50° North of West and vehicle *B*, 36 m in 60° East of North. The initial skid distance of the vehicles *A* and *B* are 38 m and 20 m respectively before collision. Find out the original speeds of the vehicles, if the average skid resistance of the pavement is found to be 0.4.**

Solution:

We will be solve this problem by using the standard equation derived in the previous discussed cases.

$$v_{A3} = \sqrt{254 fs_{A2}} = \sqrt{254 \times 0.4 \times 15} = 39.04 \text{ kmph}$$

$$v_{B3} = \sqrt{254 \times 0.4 \times 36} = 60.48 \text{ kmph}$$

Using equation 5.13 and 5.14, speeds of vehicles just before collision,

$$v_{A2} = \frac{W_B}{W_A} v_{B3} \sin B - v_{A3} \cos A$$

$$= \frac{6}{4} \times 70.9 \times \sin 60 - 45.8 \cos 50 = 62.66 \text{ kmph}$$

$$v_{B2} = \frac{W_B}{W_A} v_{A3} \sin A + v_{B3} \cos B$$

$$= \frac{4}{6} \times 45.8 \sin 50 + 70.9 \cos 60 = 58.34 \text{ kmph}$$

Original speeds of vehicles before application of brakes are obtained using equation 5.19

$$v_{A1} = \sqrt{254 fs_{A1} + v_{A2}^2} = \sqrt{254 \times 0.4 \times 38 + 62.66^2} = 88.24 \text{ kmph}$$

$$v_{B1} = \sqrt{254 fs_{B1} + v_{B2}^2} = \sqrt{254 \times 0.4 \times 20 + 58.84^2} = 74.12 \text{ kmph}$$

Thus the original speeds of vehicles *A* and *B* before the application of brakes are 88.24 and 74.12 kmph respectively.

4.4 Traffic Control Device

Traffic control device is the medium used for communicating between traffic engineer and road users. These devices are used to control, regulate and guide traffic. The generally requirements of traffic control devices are:

(i) The control device should fulfill a need: Each device must have a specific purpose for the safe and efficient operation of traffic flow.

(ii) It should command attention from the road users: This affects the design of signs. For commanding attention, proper visibility should be there. Also the sign should be distinctive and clear.

(iii) It should convey a clear and simple meaning: Clarity and simplicity of message is essential for the driver to properly understand the meaning in short time. The use of color, shape and legend as codes becomes important in this regard.

(iv) Road users must respect the signs: Respect is commanded only when the drivers are conditioned to expect that all devices carry meaningful and important messages.

(v) The control device should provide adequate time for proper response from the road users: This is again related to the design aspect of traffic control devices. The sign boards should be placed at a distance such that the driver could set it and gets sufficient time to respond to the situation.

A number of mechanisms are used by the traffic engineer to communicate with the road users. These mechanisms recognize certain human limitations, particularly eyesight. Messages are conveyed through the following elements:

(*i*) Colour (*ii*) Shape
(*iii*) Pattern (*iv*) Legend

The most common control devices are:

(*i*) Traffic signs (*ii*) Signals
(*iii*) Markings (*iv*) Islands

4.4.1 Traffic Signs

A traffic sign is a device mounted an a fixed or portable support whereby a specific message is conveyed by means of words or symbols. Traffic signs should be placed such that they could be seen and recognized by the road users easily and in time.

On the kerb roads, the edge of the sign adjacent to the road should not be less than 0.6 m away from the edge of the kerb. On roads without kerbs, the nearest edge may be 2.0 to 3.0 m from the edge of the carriageway.

The traffic signs should be mounted on sign posts painted alternately with 25 cm black and white bands. The reverse side of all the sign plates should be painted gray. Traffic signs are divided into 3 categories

(*i*) Regulatory signs (*ii*) Warning signs (*iii*) Informatory signs

Regulatory Signs

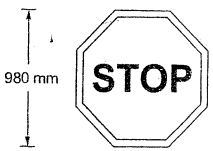
The regulatory or mandatory signs are used to inform the road users of certain laws and regulations to provide safety and free flow to traffic. The violation of these signs is a legal offence. The regulatory signs are classified under the following categories.

(i) Right of Way Series:

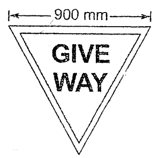
(a) It includes two unique signs i.e. stop and giveway that assign the right of way to the selected approaches of an intersection.

(b) Stop sign is intended to stop the vehicles on a roadway. It is octahedral in shape and red in colour with a white border.

(c) Give way sign is used to control the vehicles on a road so as to assign right of way to traffic on other roadways. This sign is in triangular shape with apex downwards and white in colour with a red border



STOP

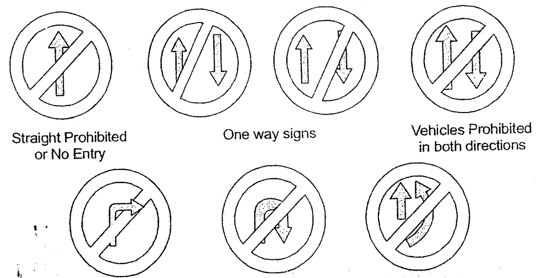


GIVE WAY

(ii) Movement Series:

(a) These are meant to prohibit certain traffic movements.

(b) They are circular in shape and white in colour with a red border.



Straight Prohibited or No Entry

One way signs

Vehicles Prohibited in both directions

Right Turn Prohibited

U-Turn Prohibited

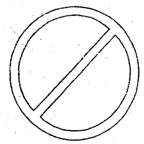
Over Taking Prohibited

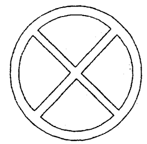
(iii) Parking Series:

(a) They are meant to prohibit parking and stopping verticals at that place.

(b) These are circular in shape with a blue background, and a red border.

(c) In no parking sign an oblique red bar at an angle of 45°. While in no stopping sign two oblique red bars at 45° and right angles to each other.


NO PARKING

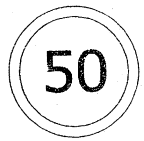

NO STOPPING

(iv) Speed Series:

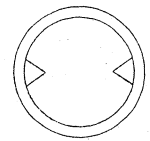
(a) Speed Limit signs are meant to restrict the speed of all or certain classes of vehicles on a particular stretch of a road.

(b) These signs are circular in shape and white background, red border and black numerals indicating the speed limit.

(c) The vehicle control signs are circular in shape, red border and black symbols instead of numerals.



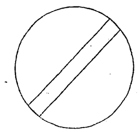
SPEED LIMIT


WIDTH LIMIT

(v) Restriction Ends Sign:

(a) They indicates the point at which are the prohibitions notified by prohibitory signs for moving vehicles binding to apply.

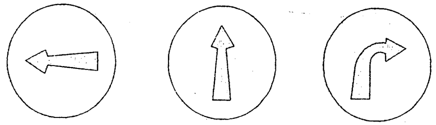
(b) They are circular in shape with white back ground and a broad diagonal black band at 45°.


RESTRICTION ENDS

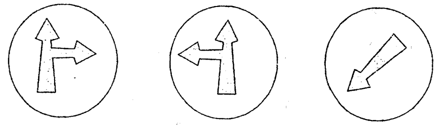
(vi) Compulsory Direction Control Signs:

(a) They are indicated by arrows, the appropriate directions in which the vehicles are bound to proceed.

(b) These are circular in shape with a blue background and white direction arrows.



Compulsory Turn left ahead | Compulsory ahead only | Compulsory Turn right ahead

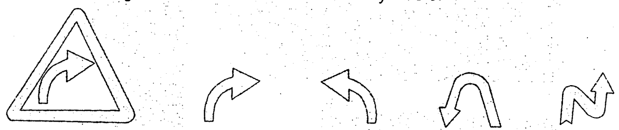


Compulsory Ahead or turn right | Compulsory Ahead or turn left | Compulsory keep left

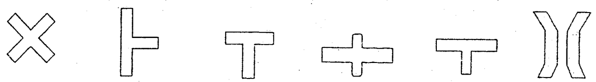
Warning Signs

These signs are used to warn the roads users at sufficient distance in advance about the impending road condition. Warning signs are also known as cautionary signs. The warning signs are in the shape of equilateral triangle with its apex pointing in upward direction.

They have a white background, red border and black symbols.



Typical cautionary sign | Right hand curve | Left hand curve | Hair pin bend left | Reverse bend right



Cross road | Side road right | T-Intersection | Major road ahead | Narrow bridge

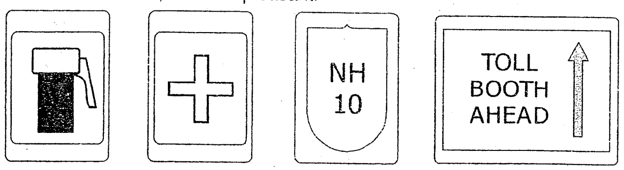
Warning distances on the basis of class of roads are shown below in Table 4.2.

Table-4.2 : Warning distances

Class of Roads	NH/SH	MDR	ODR	VR	URBAN ROADS
Distance (in m)	120	90	60	40	50

Informatory Signs

Informatory signs are provided to guide the road users about the routes, destination and to provide information which make travel easier, safe and pleasant.



The information signs are of following types:

(i) Direction and place identification signs: These are rectangular in shape with white background, black border, black arrows and black letters. They include destination signs, direction signs, route marker and place identification signs.

(ii) The facility information signs: They are rectangular in shape with blue background and white/black letters / symbols. Information signs include public telephone, Petrol Pump, Hospital, First Aid Post etc.

(iii) Parking signs: They are set up parallel to the road using square sign board with blue background and white coloured letter '*P*'.

(iv) Flood gauge sign: It is installed at all cause ways.

Design of Traffic Signs

The effectiveness of sign depends upon:

(*i*) Uniformity of design, application and installation.

(*ii*) Attention or target value depends upon the size, shape, colour, colour contrast, and level of illumination.

(*iii*) Priority value, i.e., the characteristics which determine the order in which the signs are reads. It will depend primarily on the placement and size of message compared with all others in a group.

(*iv*) Legibility which may be either pure legibility or glance legibility depending upon the conditions. Pure legibility is the distance at which the sign can be read in unlimited time. Glance legibility is the distance at which sign can be read under normal traffic conditions and speed. Both are dependent upon the letter size, shapes, level of illumination, etc. and familiarity of the motorist with the particular design.

IRC recommendations for the design of traffic signs:

(*i*) Use large signs on high-speed roads.

(*ii*) Wider spacing between letters, with optically equal spacing, depending upon the type of adjacent strokes, increases the legibility.

(*iii*) Use a maximum of three words.

(*iv*) Reflectize of illuminate the signs to be read at night.

(*v*) Location of the signs will depend on the speed of vehicles and the size of letters on the sign.

(*vi*) Keep uniformity in (a) design i.e. shape, colour (b) size of sign (c) symbols (d) word messages (e) illumination (f) lettering.

(*vii*) Distraction or advertisement signs and other unnecessary signs should be eliminated whenever possible.

(*viii*) It is recommended that two signs for different purposes should not be placed on the same sign post but should be separated by at least 30 m if possible.

(*ix*) Location of the signs with respect to the carriageway.

4.4.2 Traffic Signals

A traffic signal is defined as any power operated traffic control device or a sign by which traffic is warned or directed to take some specific action. Traffic signals is a device which is used to direct the traffic to stop and proceed at intersections using red and green traffic light signals automatically

Advantages of Traffic Signals:

(*i*) They provide orderly movement of traffic and increase the traffic handling capacity of most of the intersections at grade.

(*ii*) They reduce certain types of accidents mainly the right angled collisions.

(*iii*) In the effective use of signal system, reasonable speed is maintained along the major road traffic.

(*iv*) Signals provide a chance to crossing traffic of minor road to cross path of continuous flow of traffic stream at reasonable intervals of time.

(*v*) Automatic traffic signal may work out to be economical when compared to manual control.

Disadvantages of Traffic Signals:

(*i*) Improper design and location of signals may load to violations of the control system

(*ii*) Failure of the signal due to electric power failure or any other defect may cause confusion to the road users.

Type of Traffic Signals

(i) Traffic Control Signals: Traffic Control Signals have three coloured light glows facing each direction of traffic flow. The red light is meant for "Stop", Green light indicates "GO" and the amber light allows the clearance time for the vehicles which enter the intersection area by the end of green time. Traffic control signals are of three types:

(a) Fixed time signal

(b) Manually operated signal

(c) Traffic actuated (automatic) signal

In Fixed Time Signal the timing of each phase of the cycle is predetermined base on the traffic studies. The main drawback of this is that some times the traffic flow on one road may be almost nil and traffic on cross road may be quite heavy but signal operates with fixed timings. Traffic actuated signals are those in which the timings of phase and cycle are changed according to traffic demand.

(ii) Pedestrian Signal: It is used to give the right of way to pedestrians to cross a road when the vehicular traffic shall be stopped by stop signal.

(iii) Special Traffic Signal: Special traffic signal such as "FLASHING BEACONS" are meant to Warn the traffic. When signal is flashing red then the vehicles shall stop before entering the nearest crosswalk at an intersection.

While flashing yellow signals are caution signals meant to signify that drivers may proceed with caution.

Types of Traffic Signal System

There are four general types of signal system:

(i) Simultaneous System: In this system all the signals show the same indication at the same time. As the division of cycle is also the same at all intersections, this system does not work satisfactorily.

(ii) Alternate System: It shows opposite indications in a route at the same time and this system is more satisfactory than the simultaneous system.

(iii) Simple Progressive System: A time schedule is made to permit, as nearly as possible a continuous operation of groups of vehicles along the main road at a reasonable speed. The phases and intervals at each signal installation may be different but each signal unit works as fixed time signal with equal signal cycle length.

(iv) Flexible Progressive System: In this system we can vary the length of cycle, cycle division and the time schedule at each signalized intersection automatically with the help of a computer. It is the most efficient system.

Elements used in the Design of Traffic Signals

(i) Cycle Length (C_o): It indicates the time interval between starting of signal such as green to the next time green Starts.

(ii) Interval: It indicates the change from one stage to another. There are two types of interval:

(a) Change interval: It is also called yellow time and it indicates the interval between green and red Signal.

(b) Clearance interval: It is also known as all red time and is included after each yellow interval indicating a period during which all signal phases shows red and it is used for clearing of vehicles at the intersections.

Total cycle length = Green interval + Red interval + Change interval

NOTE: Green interval is the actual duration for which green light of traffic is turned on.

(iii) Phase: A phase is taken as the sum of green interval and clearance interval. During green interval, non conflicting movement are assigned into each phase. It allows a set of movement to flow and safely halt the flow before phase of another set of movement starts.

Two phases of 2 phase system are shown in Figure 4.12.

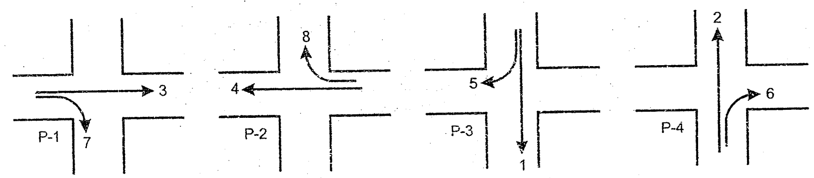
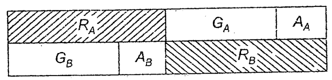


Figure-4.12: One way of providing four phase signals

Representation of intervals for 2-phases system is shown below:



NOTE: When straight moving traffic and turning traffic are comparable then 4-phase signal is adopted.

(iv) Lost Time: It represents the time during which the intersection is not effectively utilised for any movement.

Example: When the signal for an approach turns red to queue will take some time to perceive the signal and same time is lost before they moves.

Let, there is a group of '*N*' vehicles at an intersection. The 1st headway is the time interval between the initiation of the green signal and the instant vehicle crossing the kerb line.

e_i = Difference between actual and saturation headway for i^{th} vehicle.

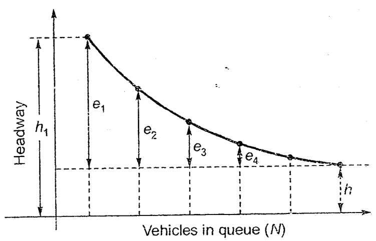


Figure-4.13: Headways of vehicles at intersection

h_i = Actual time headway of i^{th} vehicle

h = Saturation headway

2nd headway will be comparatively lower, because the second vehicle crossing the kerb line. The 1st headway is relatively longer since it includes the reaction time of the driver and the time necessary to accelerate.

Total time lost in a cycle length is given as,

$$\text{Lost time } (l) = \sum_{i=1}^{n} e_i$$

$$\text{Green Time} = l + Nh \qquad \text{(where, } N \text{ is the number of vehicles)}$$

Total time lost in a cycle length is the sum of time lost in starting delay for single phase and clearance time lost for single phase.

(v) Effective Green Time (g_i): Effective green time is the actual time available for vehicle to cross the intersection.

$$g_i = G_i + A_i - t_L$$

where, G_i = Actual green time, A_i = Amber time

t_L = Lost time

(vi) Lane Capacity: If every vehicle requires 'h' seconds time headway to cross the kerb line and assume the signal is always green then saturation capacity is given as

$$\text{Saturation capacity,} \quad S = \frac{3600}{h} \text{ veh/ hr/lane}$$

and traffic capacity of a lane $= S \times \dfrac{g_i}{C_0}$ veh/hr

where, $\dfrac{g_i}{C_0}$ = Green Ratio

C_0 = Cycle length

g_i = Effective green time

NOTE: When $g_i = c_0$, green ratio will become equal to 1 and traffic capacity of lane will be maximum.

Example-4.9 Let the cycle time of an intersection is 60 sec, green time = 27 sec, amber time = 4 sec. If saturation headway is 2.4 sec per vehicle, the start-up lost time is 2 sec per phase and clearance lost time is 1 sec per phase. Find the capacity of lane.

Solution:

Cycle length, C_0 = 60 sec

Effective green time, $g_i = G_i + A_i - t_L$

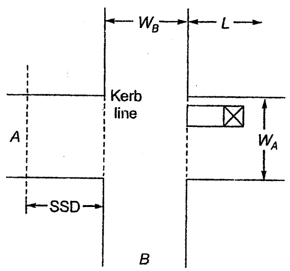
$g_i = 27 + 4 - 1 - 2 = 28$ sec

$$\text{Green ratio} = \frac{g_i}{C_0} = \frac{28}{60} = 0.467$$

$$\text{Traffic capacity of a lane} = S \times \frac{g_i}{C_0} = \frac{3600}{h} \times \frac{g_i}{c_0} = \frac{3600}{2.4} \times 0.467$$

Traffic capacity of a lane = 700 veh/hr

Design Principle of Signals



Suppose we have to design a 2 phase signal without any turning movements.

Red time of road A can be written as

$$R_A = G_B + A_B$$

similarly for road B, $R_B = G_A + A_A$

where, G_A/G_B = Green time of road A/B

Green time is calculated on the basis of number of vehicles at any road.

Amber time of a road is taken as the maximum calculated on the basis of different cases given below:

(*i*) When vehicles are within SSD from intersection.

$$\text{Amber time} = \frac{\text{SSD + Width of another road + Length of vehicle}}{\text{Speed of vehicle}}$$

(*ii*) When vehicles are beyond SSD from intersection

$$\text{Amber time of road} = \text{Reaction time + Braking time} = t_R + \frac{u}{a}$$

where, u = Speed of vehicle and a = Retardation

NOTE: Braking time is the actual duration of brake application.

Example-4.10 A driver travelling at a speed of 80 kmph was sighted crossing an intersection. He claimed that duration of amber display was improper and consequently a dilemma zone exists. Using following data determine whether driver claim was correct or not.

Amber time = 4.5 sec, Reaction time = 1.5 sec

Intersection width, W = 15 m, Car length = 4.6 m

Retardation = 3 m/s²

Solution:

$$\text{Coefficient of friction,} \quad f = \frac{a}{g} = \frac{3}{9.81} = 0.31$$

$$\text{SSD} = \frac{80}{3.6} \times 1.5 + \frac{(80/3.6)^2}{2 \times 9.81 \times 0.31} = 114.525 \text{ m}$$

(*i*) $$\text{Amber time} = \frac{\text{SSD} + W + L}{v} = \frac{114.525 + 15 + 4.6}{80/3.6} = 6.04 \text{ sec}$$

(*ii*) $$\text{Amber time} = \text{Reaction time + Braking time} = 1.5 + \frac{80/3.6}{3} = 8.9 \text{ sec}$$

So, amber time required is 8.9 sec and provided amber time is 4.5 sec. Hence the driver claim was right.

4.4.3 Methods of Signal Designing

There are various method of signal designing:

(*i*) Trial Cycle Method (*ii*) Approximate Method
(*iii*) Webster Method (*iv*) IRC Method

Trial Cycle Method

In this method green time is calculated on the basis of number of vehicles per hour accumulated at intersection. Steps involved in this method are as follows:

(*i*) 15 minute traffic counts on road *A* and *B* are noted as n_A and n_B during the design peak hour flow.
(*ii*) Assume a suitable trial cycle, '*C*' seconds.
(*iii*) Calculate the number of vehicles passing in one cycle time.

$$\text{for road } A = x_A = \left(\frac{n_A}{15\times 60}\right)\times C$$

$$\text{for road } B = x_B = \left(\frac{n_B}{15\times 60}\right)\times C$$

where, x_A and x_B are number of vehicles passing in one cycle time on road *A* and *B*.

(*iv*) Calculate the green period of road *A* and road *B*

$$G_A = x_A \times h = 2.5\,x_A$$
$$G_B = x_B \times h = 2.5\,x_B$$

NOTE

- Average time required for one vehicle to cross the intersection is equal to time headway which is generally taken as 2.5 seconds.
- Cycle time is generally assumed in the multiple of 5 for simplicity in calculation.

(*v*) Total cycle time can be calculated as

$$C_0 = G_A + R_A + A_A$$

or,

$$C_0 = G_A + A_A + G_B + A_B \qquad (\because R_A = G_B + A_B)$$

If the calculated cycle time C_0 is equal to the assumed cycle time C_1, then the calculated cycle time is taken as the design cycle time.

Example-4.11 The 15 minute-traffic counts on cross roads *A* and *B* during peak hour are observed as 178 and 142 vehicles per lane respectively approaching the intersection in the direction of heavier traffic flow. If the amber times required are 3 and 2 seconds respectively for two roads based on approach speeds, design the signal timings by trial cycle method. Assume an average time headway of 2.5 seconds during green phase.

Solution:

Trial-1

Assume a trial cycle $C_1 = 50$ sec

$$\text{Number of cycles in 15 mins} = \frac{900}{50} = 18$$

Green time for road *A*, according to an average time headway of 2.5 sec

$$G_A = \frac{178\times 2.5}{18} = 24.72 \text{ sec}$$

Green time for road *B*, $$G_B = \frac{142\times 2.5}{18} = 19.72 \text{ sec}$$

Amber times A_A and A_B are 3 and 2 sec

∴ Total cycle length = 24.72 + 19.72 + 3.0 + 2.0 = 49.44 sec

As this is lower than the assumed trial cycle of 50 sec, another lower cycle length may be tried.

Trial-2

Assume a trial cycle $C_2 = 40$ sec

$$\text{Number of cycles in 15 minutes} = \frac{900}{40} = 22.5$$

$$\text{Green time for road } A,\ G_A = \frac{178\times 2.5}{22.5} = 19.78 \text{ sec}$$

$$\text{Green time for road } B,\ G_B = \frac{142\times 2.5}{22.5} = 15.78 \text{ sec}$$

∴ Total cycle length = 19.78 + 15.78 + 3.0 + 2.0 = 40.56 sec

Trial-3

Assume a trial cycle $C_3 = 45$ sec

$$\text{Number of cycles in 15 minutes} = \frac{900}{45} = 20$$

Green time for road *A*, $$G_A = \frac{178\times 2.5}{20} = 22.25 \text{ sec}$$

Green time for road *B*, $$G_B = \frac{142\times 2.5}{20} = 17.75 \text{ sec}$$

∴ Total cycle length = 22.25 + 17.75 + 3.0 + 2.0
= 45.0 sec

Therefore the trial cycle of 45 secs. may be adopted with the following signal phases:
$G_A = 22.25$, $G_B = 17.75$, $A_A = 3.0$, $A_B = 2.0$ and Cycle length = 45.0 sec

Approximate Method

In this method cycle length depends on the pedestrian time to cross the road and traffic volume. Steps involved in this method are:

(*i*) Calculate the time required by pedestrian to cross road *A/B* which is also called as clearance interval.

$$\text{Clearance Interval}, \quad C.I = \frac{W_A/W_B}{1.2 \text{ m/s}}$$

(Here, 1.2 m/s is considered as the walking speed of pedestrian)

(*ii*) Minimum green time for pedestrian to cross road *A/B*

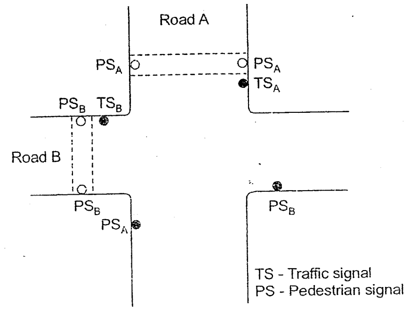


***Figure-4.14**: Placement of signals*

$$G_{PA}/G_{PB} = C.I_{A/B} + \text{Initial walking period of 5 sec}$$

$$G_{PA} = R_A = G_B + A_B$$

$$G_{PB} = R_B = G_A + A_A$$

(*iii*) If n_A and n_B are the approaching volume of heaviest traffic per hour per lane on road A and road B respectively, then

$$\frac{G_A}{G_B} = \frac{n_A}{n_B}$$

(*iv*) Calculate the total cycle length

$$C_0 = G_A + A_A + R_A$$

$$C_0 = G_A + A_A + G_B + A_B \qquad (\because R_A = R_B + A_B)$$

(*v*) Do not walk time for pedestrian is given as

$$DNW_A = G_A + A_A = R_B$$

Similarly, $$DNW_B = G_B + A_B = R_A$$

NOTE: If pedestrian signal is used, then the minimum 7 seconds walking period should be provided.

Webster Method

In this method the optimum cycle length (C_0) calculated on the basis of least total delay to the vehicles at the signalized intersection.

$\therefore$ Optimum cycle time, $C_0 = \frac{1.5L+5}{1-Y}$ seconds

where, L = Total time lost = $nt_L + R$ or $L = n(t_{SL} + t_{CL}$ + All red time)

t_{SL} = Start-up lost time

t_{CL} = Clearance lost time

n = Number of phase

R = All red time required for pedestrian to cross the road i.e. generally taken as 16 sec.

For the average signal cycle, the lost time (t_L) is taken to be 2 seconds.

$\therefore$ Total time lost, $L = 2n + R$

and, $Y = y_A + y_B$

$$y_A = \frac{q_A}{S_A} \text{ and } y_B = \frac{q_B}{S_B}$$

where, q_A/q_B = Normal flow in road A/B in veh/hr/lane

S_A/S_B = Saturation flow in road A/B in veh/hr/lane

$\therefore$ Green time for road A is given by, $G_A = \frac{y_A}{Y}(C_0 - L)$ sec

and for road B, $G_B = \frac{y_B}{Y}(C_0 - L)$ sec

Example-4.12 Design two phase traffic signal by Webster's method using the following data:

Road	Average Normal Flow (in veh/hr)	Saturation Flow (in veh/hr)
A	400	1250
B	250	1000

Take all red-time required for pedestrian crossing is 12 sec and amber time of 2.0 sec for each lane.

Solution:

$$y_a = \frac{q_a}{S_a} = \frac{400}{1250} = 0.32$$

$$y_b = \frac{q_b}{S_b} = \frac{250}{1000} = 0.25$$

$$Y = y_a + y_b = 0.32 + 0.25 = 0.57$$

$$L = 2n + R = 2 \times 2 + 12 = 16 \text{ sec}$$

$$C_0 = \frac{1.5L+5}{1-Y} = \frac{1.5\times16+5}{1-0.57} = \frac{29}{0.43} \simeq 67.5 \text{ sec}$$

$$\therefore \quad G_a = \frac{y_a}{Y}(C_0 - L) = \frac{0.32}{0.57}(67.5-16) \simeq 29 \text{ sec}$$

$$G_b = \frac{y_b}{Y}(C_0 - L) = \frac{0.25}{0.57}(67.5-16) \simeq 22.6 \text{ sec}$$

$\therefore$ Total cycle time = 29 + 22.6 + 12 + 4 = 67.6 sec

IRC Method

It is a combination of approximate and webster method. In IRC method signal timing is decided by approximate method and design is checked by webster method.

Following steps are taken in the IRC method are:

(*i*) First calculate the cycle length with the help of approximate method.

(*ii*) Check the calculated cycle length by the IRC method.

Minimum Green time is obtained by assuming that first vehicle will take 6 seconds and subsequent will be clear at the rate of 2 seconds. Then green time for road A and B are:

$$G_A = 1 \times 6 \text{ sec} + (x_A - 1)\, 2 \text{ sec} \nless 16 \text{ sec}$$

and $$G_B = 1 \times 6 \text{ sec} + (x_B - 1)\, 2 \text{ sec} \nless 16 \text{ sec}$$

where, x_A and x_B are number of vehicles in Road A and Road B in cycle length.

(*iii*) Check by webster method, green time calculated by webster method should not be more than green time by approximate method.

Green time from IRC method should not be more than green time from approximate method.

4.5 Intersection

Intersection is the area where two or more roads meet. At intersection there are through, turning and crossing traffic and their movements may be controlled in different ways depending on the type of intersection and its design. Its main function is to guide vehicles to their respective directions.

Due to movement of traffic at intersection various types of conflicts occur like crossing, merging and diverging conflict. Generally merging from right and diverging to right creates conflict. Consider a typical four-legged intersection as shown below.

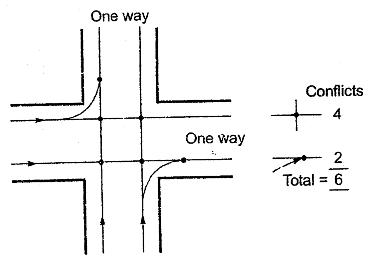


Figure-4.15 : *Vehicular conflicts with one-way regulation on both roads*

Conflict points for crossing movements are 4, for diverging movement are 4, for merging movement are 4 and for weaving movements are 12. Hence, there are total 24 types of vehicular conflict points.

In a typical four legged intersection there are 8 pedestrian conflict points also. Hence total 32 different types of conflicts points are formed in a four legged intersection.

Crossing conflicts are the major conflicts and merging and diverging conflicts are minor conflicts. To reduce the conflicts at intersection we have to control it effectively

Table-4.3 : Conflicts points

Number of lanes		Number of potential conflicts		
Road A	Road B	Both roads Two way	A-One way B-Two way	Both roads One way
2	2	24	11	6
2	3	24	11	8
2	4	32	17	10
3	3	24	13	11
4	4	44	25	18

Various types of intersection controls are discussed below:

(i) Passive Control: It is used when volume of traffic is less and Road sign and road marking are used to control the traffic on minor road to slow down and allow that an major road to proceed. In this control system road users are required to follow traffic rules.

(ii) Semi Control: This control system guides the driver gently to avoid conflict. Channelization and rotary are two example of this.

(iii) Active Control: In this the road users are forced to follow the path suggested by traffic control agencies. Traffic signals and grade separated intersection come under this classification.

4.5.1 Types of Intersection

(i) Intersection at Grade: All road intersections which meet at the same level allowing traffic movements like merging, diverging, crossing and weaving are called intersection at grade. These intersection are further classified as unchannelized, channelized and rotary intersections

Basic requirements of Intersection at grade are:

(a) At the intersection the area of conflict should be as small as possible.

(b) The relative speed and particularly the angle of approach of vehicle should be small.

(c) Adequate visibility should be available for vehicles approaching the intersection

(d) Sudden change of path should be avoided.

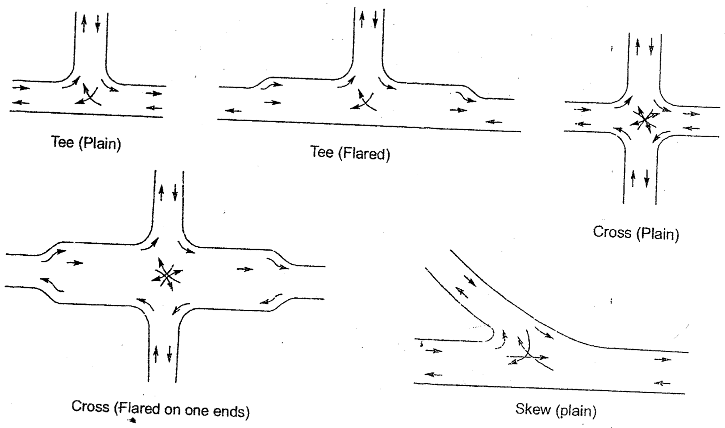


***Figure-4.16 :** Unchannelized intersection*

Unchannelized Intersections: In this type of intersection area is paved and there is absolutely no restriction to vehicles to use any part of intersection area. When no additional pavement width for turning movement is provided, it is called plain intersection. When the pavement is widened at the intersection area, by a traffic lane or more, it is known as flared intersection.

The conflict area is quite large as path of turning vehicles are not restricted or controlled then one of the crossing vehicle will have to stop while the other proceeds.

Channelized Intersections: Channelized intersection is achieved by introducing islands into the intersection area, thus reducing the total conflict area available in the unchannelized intersection.

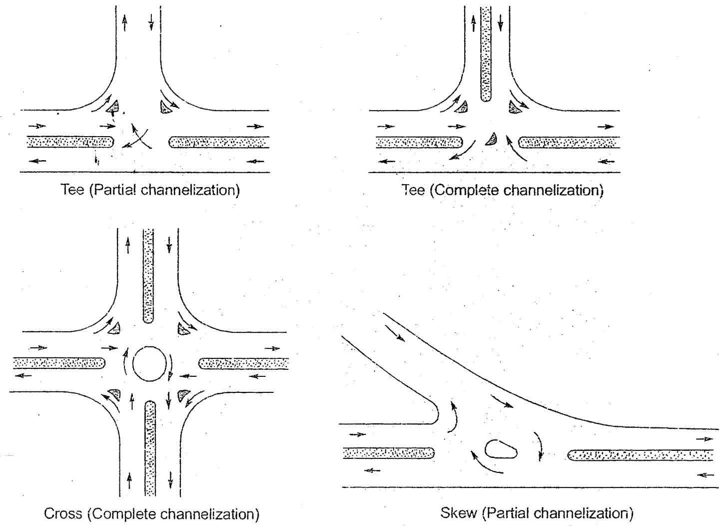


***Figure-4.17 :** Channelized intersections*

It is very useful as traffic control devices for intersection at grade and when the direction of the flow is to be changed.

Rotary Intersection: It is an enlarged road intersection where all converging vehicles are forced to move round a large central island in clockwise direction before they can weave out of traffic flow into their respective directions.

(ii) Grade Separated Intersection: Grade separation structures that permit the cross flow of traffic at different levels without interruptions.

Advantages of grade separation are:

(a) There is increased safety for turning traffic and by indirect interchange ramp even right turn movement is quite easy and safe.

(b) There is overall increase in comfort and convenience to the road users.

(c) Stage constructions of additional ramps are possible after the grade separation structure between main roads are constructed.

Disadvantages of grade separation are:

(a) It is very costly to provide complete grade separation and interchange facilities.

(b) Construction of grade separation is difficult and undesirable in the area where there is limited right of way.

(c) In flat or plain terrain, grade separation may introduce undesirable sags in the vertical alignment.

(iii) Traffic Islands: Traffic islands are constructed within the roadway to establish physical channels through which the vehicular traffic may be guided. Classification of traffic islands on the basis of there function are:

(a) **Divisional Islands:** Pedestrian loading islands are provided as regular bus stops and similar places for the protection of passengers. A pedestrian islands at or near a cross walk to aid and protect pedestrian crossing the carriageway.

(b) **Channelizing Islands:** Channelizing islands are used to guide the traffic into proper channel through the intersection area. These are very useful as traffic control devices for intersection at grade.

(c) **Pedestrian Loading Islands:** Divisional islands are supposed to separate opposing flow of traffic on a highway with four or more lanes. By thus head on collisions are eliminated.

(d) **Rotary:** Rotary island is the large central island of a rotary intersection. The crossing movement is converted to weaving by providing sufficient weaving length.

NOTE: The area adjacent to the kerb which is kept reserved for use by stopped bus may be called as bus kerb loading zone.

(iv) Interchange: An interchange is a grade separated intersection with the facilities of ramp for turning traffic between approaching highway.

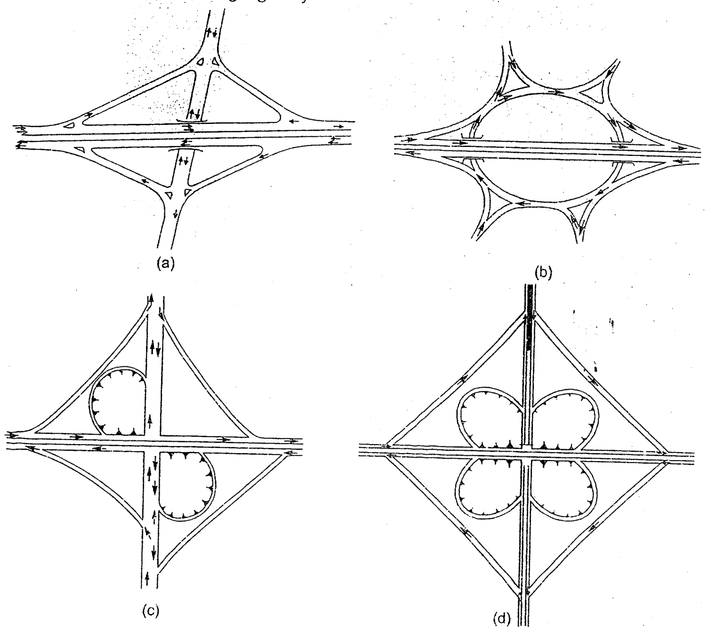


Figure-4.18 : Types of Interchanges (a) Diamond (b) Rotary interchanges (c) Partial clover leaf and (d) Full clover leaf

4.6 Traffic Rotaries

Rotary intersections are special form of intersection at grade laid out for the movement of traffic in one direction around a central traffic island. The vehicles entering the rotary are gently forced to move in a clockwise direction in orderly fashion. They then weave out of the rotary to the desired direction.

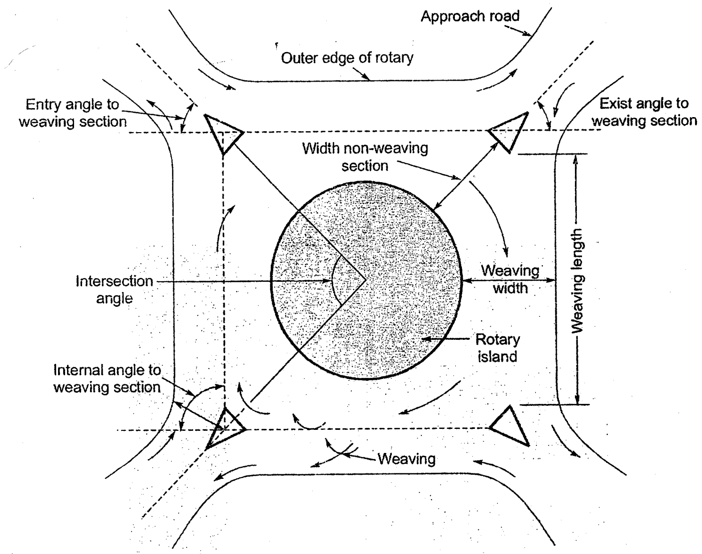


Figure-4.19 : Rotary elements

General Guidelines for the selection of rotaries are:

(*i*) Rotaries are suitable when the traffic entering from all the four approaches are relatively equal.

(*ii*) A total volume of about 3000 vehicles per hour can be considered as the upper limiting case and a volume of 500 vehicles per hour is the lower limit.

(*iii*) A rotary is very beneficial when the proportion of the right turn traffic is very high, typically if it is more than 30 percent.

(*iv*) Rotaries are suitable when there are more than four approaches or if there is no separate lanes available for right turn traffic. Rotaries are ideally suited if the intersection geometry is complex.

Advantages of Rotary:

(*i*) Traffic flow is regulated to only one direction of movement, thus eliminating severe conflicts between crossing movements.

(*ii*) All the vehicles entering the rotary are gently forced to reduce the speed and continue to move at slower speed. Thus none of the vehicles need to be stopped, unlike in a signalized intersection.

(*iii*) Because of lower speed of negotiation and elimination of severe conflicts, accidents and their severity are much less in rotaries.

(*iv*) Rotaries are self governing and do not need practically any control by police or traffic signals.

(*v*) They are ideally suited for moderate traffic, especially with irregular geometry, or intersections with more then three or four approaches.

Disadvantages of Rotary:

(*i*) All the vehicles are forced to slow down and negotiate the intersection. Therefore, the cumulative delay will be much higher than channelized intersection.

(*ii*) Even when there is relatively low traffic, the vehicles are forced to reduce their speed.

(*iii*) Rotaries require large area of relatively flat land making them costly at urban areas.

(*iv*) The vehicles do not usually stop at a rotary. They accelerate and exit rotary at relatively high speed. Therefore, they are not suitable when there is high pedestrian movements.

4.6.1 Shape of Rotary Island

There are many different shapes of rotary required on the basis of volume of traffic approaching.

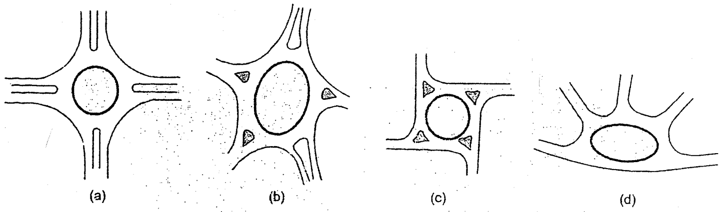


***Figure-4.20**: Shapes of rotary islands (a) Circular (b) Elliptical (c) Turbine and (d) Tangent*

Circular shape of rotary is preferred when traffic in both the roads are comparable and tangential shape of rotary is prefered when traffic in one direction is significant compared to traffic in other direction.

4.6.2 Design Speed of Rotary

At rotary all the vehicles are required to slowdown their speed. So the design speed of rotary is generally much lower than approaching roads. As per IRC the design speed for rural road is taken as 40 kmph and 30 kmph for the urban roads.

4.6.3 Radius of Curve at Entry

Radius at entry depends on various factors like design speed, superelevation and coefficient of lateral friction. The entry to rotary is not straight but small curvature is introduced which force the driver to reduce the speed.

Rotary design speed (kmph)	Suggested values of radius at entry (m)
40	20 - 35
30	15 - 25

4.6.4 Radius of Curve at Exit

Exit radius should be higher than entry radius and radius of rotary island so that vehicle discharge from the rotary at a higher rate

$$R_{exit} = (1.5 - 2)\, R_{entry}$$

4.6.5 Radius of Central Traffic Island

Theoretically the radius of central island should be equal to the radius at entry but in practise it is normally kept slightly greater than radius at entry. Radius of central island is taken to be 1.33 times the radius at entry

$$R_{C.I} = 1.33 \times R_{entry}$$

4.6.6 Width of Weaving Section (*w*)

Entry and exit width of the roadway is governed by the traffic entering and leaving the intersection and the width of approaching road. The width of carriageway at entry and exit will be lower than width of carriageway at the approaches to enable reduction of speed.

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

where, e_1 = Entry width and e_2 = Exit width

Approach road width (m)	7.0	10.5	14
Entry width, e_1 (m)	6.5	7.0	8.0

NOTE: Take width at exit as entry width if nothing is given about the exit width.

4.6.7 Weaving Length (*L*)

Weaving length determines how smoothly the traffic can merge and diverge. Very large weaving length is also dangerous as it may encourage over speeding. Length of weaving section is kept atleast 4 times the width of the weaving section.

To discourage overspeeding in the weaving section s, the maximum weaving length should not exceed the above given values.

Design Speed (kmph)	Minimum Weaving Length (m)
40	45
30	30

4.6.8 Entry and Exit Angles

Entry angle should be larger than exit angle and desirable entry angle is 60°. Exit angles should be small and 30° is desired exit angles.

4.6.9 Capacity of the Rotary

Capacity of a rotary is determined by the capacity of each weaving section. Capacity of individual weaving sections depends upon

(*i*) Width of weaving section (*w*)

(*ii*) Average width of entry into rotary (*e*)

(*iii*) Weaving length (*l*)

(*iv*) Proportioning ratio (*p*)

Practical capacity of rotary in PCU per hour is given by

$$Q = \frac{280w\left(1+\frac{e}{w}\right)\left(1-\frac{p}{3}\right)}{1+\frac{w}{l}} \quad \text{...(i)}$$

Calculation for proportioning ratio (*p*),

$$p = \frac{\text{Crossing/ Weaving Traffic}}{\text{Total Traffic}}$$

$$p = \frac{b+c}{a+b+c+d}$$ (proportioning ratio always lies between 0.4 and 1)

Where,
- a = Left turning traffic moving along left extreme lane
- b = Crossing / Weaving traffic turning towards right while entering to the rotary
- c = Crossing / Weaving traffic turning towards left while leaving rotary
- d = Right turning traffic moving along right extreme lane

Conditions to apply the above formula of capacity of rotary are:

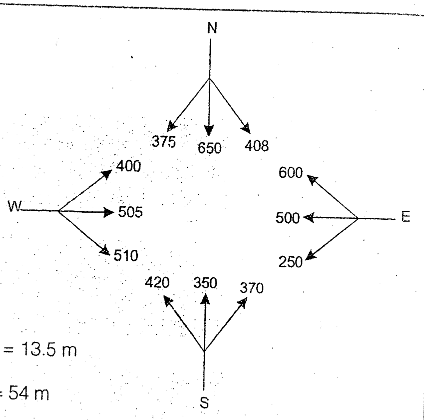
(i) 6 m ≤ width of weaving section (w) ≤ 18 m (ii) $\frac{e}{w} = (0.4 - 1)$

(iii) $\frac{w}{L} = 0.12 - 0.4$ (iv) $p = (0.4 - 1)$

(v) $l = 18 - 90$ m

Illustrative Examples

Example-4.13 The width of a carriage way approaching an intersection is given as 15 m. The entry and exit width at the rotary is 10 m. The traffic approaching the intersection from the four sides is shown in the figure. Find the capacity of the rotary using the given data.



Solution:

$$e_1 = e_2 = 10 \text{ m}$$

(i) Weaving Width (w) $= \left(\frac{e_1 + e_2}{2}\right) + 3.5$

$$= \left(\frac{10+10}{2}\right) + 3.5 = 13.5 \text{ m}$$

(ii) Weaving length, $l = 4\,w = 4 \times 13.5 = 54$ m

(iii) Calculation for proportioning ratio (p)

$$p_{ES} = \frac{\text{Weaving Traffic}}{\text{Total Traffic}} = \frac{510 + 650 + 500 + 600}{510 + 650 + 500 + 600 + 250 + 375}$$

$$= \frac{2260}{2885} = 0.783$$

$$p_{W-N} = \frac{550 + 510 + 350 + 600}{505 + 510 + 350 + 600 + 400 + 370} = \frac{1965}{2735} = 0.718$$

$$p_{N-E} = \frac{650 + 375 + 505 + 370}{650 + 375 + 505 + 370 + 510 + 408} = \frac{1900}{2818} = 0.674$$

$$p_{S-W} = \frac{350 + 370 + 500 + 375}{350 + 370 + 500 + 375 + 420 + 600} = \frac{1595}{2615} = 0.6099$$

Highest proportioning ratio will give the minimum capacity

Hence, $P = p_{E-S} = 0.783$

(iv) Therefore, the capacity of the rotary will be the capacity of this weaving section

$$Q_{ES} = \frac{280 \times 13.5\left[1 + \frac{10}{13.5}\right]\left[1 - \frac{0.783}{3}\right]}{1 + \frac{13.5}{54}} = 3890.1 \text{ veh/hr}$$

Example-4.14 The intersection of Madhya marg and Udhyan path in Chandigarh is to be signalized. Madhya marg is 13 m wide, having an approach volume of 600 vehicles per hr and 70 - 30 split during the peak hour. Approach speed is 55 km/hr. Udhyan path is 7 m wide having an approach volume of 450 vehicles/hour with a 80 - 20 split and approach speed of 40 km/hr. There is a lot of pedestrian and bicycle traffic at the intersection. Determine the cycle time, green and amber times for each street.

Solution:

Step-I: Pedestrian consideration:

Time required to indicate the pedestrian to start = 5 sec

Madhya marg:

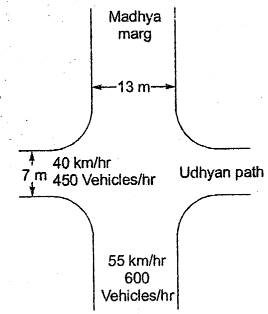
Time required to cross Madhya marg = (green + amber) time for Madhya marg = $\frac{7}{1} + 5 = 12$sec.

Udhyan path:

Time required to cross Udhyan path = (green + amber) time for Udhyan path = $\frac{13}{1} + 5 = 18$sec.

Step-II: Clearance interval : Minimum amber time for Madhya marg (speed = 55 km/hr) = 4 sec

Minimum amber time for Udhyan path (speed = 40 km/hr)= 3 sec



Step-III: Clearance interval:

Stopping sight distance for vehicles at 55 km/hr on Madhya marg = 75 m

Stopping sight distance for vehicles at 40 km/hr on Udhyan path = 50 m

Total distance to be travelled to clear intersection for vehicles moving on Madhya marg

= 75 + 7 = 82 m

Total distance to be travelled to clear intersection for vehicles moving on Udhyan path

= 50 +13 = 63 m

Time required by vehicles on Madhya marg $= \frac{82 \times 3.6}{55} = 5.7$ sec say 6 sec

Time required by vehicles on Udhyan path $= \frac{63}{40} \times 3.6 = 5.67$sec. say 6 sec

Assume the total cycle length of signals = 40 sec

Amber time for Madhya marg A_M = 6 sec

Amber time for Udhyan path A_U = 6 sec

Green time for Madhya Marg and Udhyan path = 40 – (6 + 6) = 28 sec

This time is to be divided in the ratio of approach volumes

$$\frac{G_U}{G_M} = \frac{V_U}{V_M}$$

Volume of traffic on Udhyan path/lane = $\frac{450 \times 80}{100} = 360$ vehs/hr

Volume of traffic on Madhya marg/lane = $\frac{600 \times 70}{100} \times \frac{1}{2} = 210$ vehs/hr

$$\frac{G_U}{G_M} = \frac{360}{210}; \quad G_U + G_M = 28 \text{ sec}$$

$$\frac{28 - G_M}{G_M} = \frac{12}{7} \quad \text{or} \quad \frac{28}{G_M} = \frac{19}{7}$$

$$G_M = \frac{7}{19} \times 28 = 10.32 \text{sec} \quad (\text{Take } G_M = 10 \text{ sec})$$

∴ G_U = 18 sec

For Madhya marg: G_M = 10 sec

For Udhyan path: G_U = 18 sec

Distribution of time-interval of the cycle in the two directions is as follows:

Cycle time = 40 sec (100%)

G_U = 18 sec (45%)

A_U = 6 sec (15%)

G_M = 10 sec (25%)

A_M = 6 sec (15%)

Minimum phase of (green + amber) time for Madhya marg = 10 + 6 = 16 sec

These times also satisfy the pedestrian requirements

Considering equal headway on both roads = 2.5 sec

Madhya marg

Volume entering the intersection/lane = $\frac{420}{2} = 210$ vehs/hr

Volume entering the intersection during the cycle length = $\frac{210}{60 \times 60} \times 40 = 3$ vehs/hr

Time required to clear the intersection = 3 × 2.5 = 7.5 sec

Actual time provide = 10 sec

Example 4.15 **If there is no pedestrian and cycle traffic in problem 4.1, what will be the minimum green and amber time for each street, assuming: (*i*) Equal headways on both sides and (*ii*) 4 sec headway on Madhya marg and 3 sec headway on Udhyan path.**

Solution:

(*i*) From the previous problem we note that the results were not affected by the pedestrians considerations (clearance interval and approach volume).

Assuming equal headway, the same timings can be applied, viz.,

Total cycle = 40 sec (100%)

G_U = 18 sec (45%)

= 6 sec (15%)

= 10 sec (25%)

A_M = 6 sec (15%)

(*ii*) Assuming total cycle length = 50 sec

From clearance consideration:

A_M = 6 sec

A_U = 6 sec

Green time for Madhya marg and Udhyan path = 40 – 12 = 28 sec

Madhya marg: Volume entering the intersection per cycle time = 3 vehicles

Green time required to clear the intersection = 3 × 4 = 12 sec.

Udhyan Path: Volume entering the intersection per cycle time = 4 vehicles

Green time required to clear the intersection = 4 × 3 = 12 sec

Allowing, G_U = 16 sec

Cycle time = 40 sec (100%)

G_U = 16 sec (40%)

A_U = 6 sec (15%)

G_M = 12 sec (30%)

A_M = 6 sec (15%)

Example 4.16 **A four-legged right angled intersection is to be signalised with a fixed time 2 phase signal.**

The design hour flow and saturation flow are as under

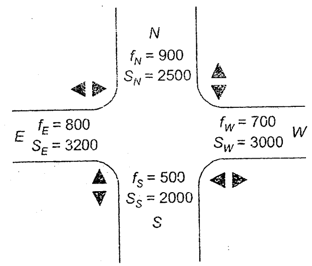
The lost time may be taken as 2 seconds per arm. Determine the optimum cycle time and apportion the green times in the two phases.

	North	South	East	West
Design hour flow in PCU/hour	900	500	800	700
Saturation flow in PCU/hour	2500	2000	3200	3000

Solution:

$$\frac{f_N}{S_N} = \frac{900}{2500} = 0.36$$

$$\frac{f_s}{S_s} = \frac{500}{2000} = 0.25$$



∴ Maximum value of f/S in the N-S direction = 0.36

$$\frac{f_E}{S_E} = \frac{800}{3200} = 0.25$$

$$\frac{f_W}{S_W} = \frac{700}{3000} = 0.33$$

∴Maximum value of f/S in the N-S direction = 0.33

L = Lost time due to starting and stopping

= 2 + 2 + 2 + 2 = 8 sec

Optimum cycle time, $$C_0 = \frac{1.5L + 5}{1 - Y} = \frac{1.5 \times 8 + 5}{1 - (0.36 + 0.33)} = \frac{17}{1 - 0.69} = \frac{17}{0.31}$$

= 54.8 sec (say 55 sec)

∴ Effective green time per cycle = $C_0 - L$ = 55 – 8 = 47 seconds

Effective green time per phase is given by

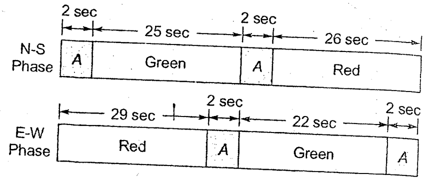
$$G_{NS} = \frac{Y_{NS}}{Y}(C_0 - L) = \frac{0.36}{0.69} \times 47$$

$$= 24.45 \text{ sec (say 25 sec)}$$

$$\therefore \quad G_{EW} = \frac{Y_{EW}}{Y} \times 47 = \frac{0.33}{0.69} \times 47$$

$$= 22.48 \text{ sec (say 22 sec)}$$

The cycle time diagrams are drawn :



Example-4.17 Calculate the basic capacity for light weight and commercial vehicles for a lane on a State Highway in plain country. Assume the length of light vehicle as 4.5 m and commercial vehicle as 6.0 m. [Take $f = 0.4$]

Solution:

The traffic lane capacity for light weight vehicles is given by

$$C_l = \frac{1000\,V}{L + 0.278\,Vt + \dfrac{V^2}{254f}}$$

For a State Highway in plain country

$$V = 100 \text{ km/hr}$$

For light weight vehicles, $L = 4.5$ m.

$$\therefore \quad C_l = \frac{1000 \times 100}{4.5 + 0.278 \times 100 + \dfrac{(100)^2}{254 \times 0.4}} = \frac{1000 \times 100}{130.725} = 755 \text{ vph}$$

Traffic capacity for commercial vehicles is given by equation

$$C_c = \frac{1000\,V}{6 + 0.278\,V + \dfrac{V^2}{170f}} = \frac{1000 \times 100}{6 + 0.278 \times 100 + \dfrac{(100)^2}{170 \times 0.4}}$$

$$= \frac{1000 \times 100}{6 + 27.8 + 147.05} = \frac{1000 \times 100}{180.86} = 553 \text{ vph}$$

Example-4.18 The average normal flow of traffic on cross roads A and B during design period are 400 and 250 pcu per hour; the saturations flow values on these roads are estimated as 1250 and 1000 pcu per hour respectively. The all-red time required for pedestrian crossing is 12 secs. Design two phase traffic signal by Webster's method.

Solution:

$$y_a = \frac{q_a}{S_a} = \frac{400}{1250} = 0.32$$

$$y_b = \frac{q_b}{S_b} = \frac{250}{1000} = 0.25$$

$$Y = y_a + y_b = 0.32 + 0.25 = 0.57$$

$$L = 2n + R = 2 \times 2 + 12 = 16 \text{ sec}$$

$$C_0 = \frac{1.5L + 5}{1 - Y} = \frac{1.5 \times 16 + 5}{1 - 0.57} = \frac{29}{0.43} = 67.4 \text{ say, } 67.5 \text{ secs}$$

$$G_a = \frac{y_a}{Y}(C_0 - L) = \frac{0.32}{0.57}(67.5 - 16) = 29 \text{ secs}$$

$$G_b = \frac{Y_b}{Y}(C_0 - L) = \frac{0.25}{0.57}(67.5 - 16) = 22.5 \text{ secs}$$

All-red time for pedestrian crossing = 12 secs

Providing Amber times of 2.0 secs each for clearance, total cycle time

= 29 + 22.5 + 12 + 4 = 67.5 secs

Example-4.19 Four legs in a rotary intersection are designated as 1, 2, 3 and 4. The traffic volumes (Vij) in terms of PCU per hour are given as:

V_{12}	150	V_{21}	310	V_{31}	1520	V_{41}	30
V_{13}	450	V_{23}	200	V_{32}	570	V_{42}	1080
V_{14}	412	V_{24}	1090	V_{34}	240	V_{43}	600

What is the proportion of weaving traffic to total traffic in the weaving section between legs 2 and 3? What is the use of this value?

Solution:

Traffic flow leaving leg 1 = $V_{12} + V_{13} + V_{14}$ = 150 + 450 + 412 = 1012

Traffic flow approaching leg 1 = $V_{21} + V_{31} + V_{41}$ = 310 + 1520 + 30 = 1860

Traffic flow leaving leg 2 = $V_{21} + V_{23} + V_{24}$ = 310 + 200 + 1090 = 1600

Traffic flow approaching leg 2 = $V_{12} + V_{32} + V_{42}$ = 150 + 570 + 1080 = 1800

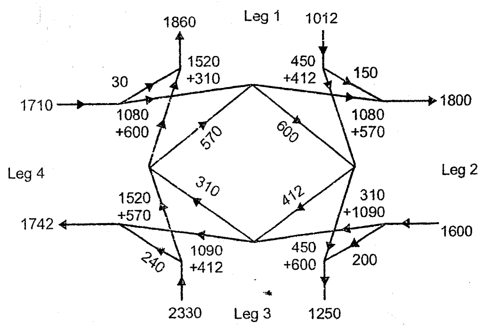
Traffic flow leaving leg 3 = $V_{31} + V_{32} + V_{34}$ = 1520 + 570 + 240 = 2330

Traffic flow approaching leg 3 = $V_{13} + V_{23} + V_{43}$ = 450 + 200 + 600 = 1250

Traffic flow leaving leg 4 = $V_{41} + V_{42} + V_{43}$ = 30 + 1080 + 600 = 1710

Traffic flow approaching leg 4 = $V_{14} + V_{24} + V_{34}$ = 412 + 1090 + 240 = 1742

The traffic assigned to the network is shown below:



Weaving traffic between legs 2 and 3 = (310 + 1090) + (450 + 600) = 2450

Total traffic between legs 2 and 3 = (310 + 1090) + (450 + 600) + 412 + 200 = 3062

$\therefore$ Proportion of weaving traffic to total traffic between legs 2 and 3 = $\frac{2450}{3062} = 0.8$

This value is also known as weaving ratio. It is useful in calculating the practical capacity of the rotary intersection. Its value should lie between 0.4 and 1.0.

Example-4.20 A road consists of 4 lanes, 2 in each direction. The maximum capacity of 2 lanes in one direction is 2000 vehicle/hour. When vehicles are stationary in a jamming condition, the average length occupied by a vehicle is 6.25 m. During a period of observation, the actual volume of traffic in one direction is steady at the rate of 1200 vehicle/hour. This flow is brought to a half when a traffic signal turns red and a queue forms. Find the time in seconds which elapses from the moment the signal turns red until the stationary queue reaches another intersection 75 m from the signal. Assume a linear relationship between speed and concentration.

Solution:

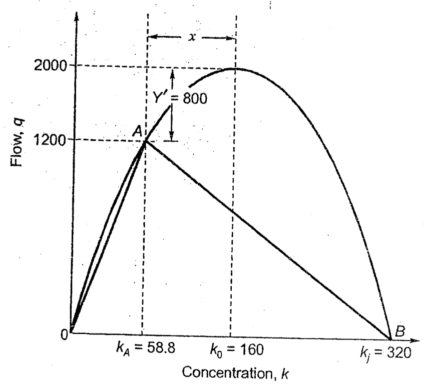
Since the relationship between speed and concentration is linear, the *q-k* curve will be parabolic.

q_{max} = 200 vehicles/hour (for 2 lanes)

$$k_j = \frac{1000}{6.25} = 160 \text{ veh/km/lane}$$

$$= 320 \text{ veh/km for 2 lanes}$$

$$q = 1200 \text{ veh/hour (for 2 lanes)}$$



Since the parabola is symmetrical about the optimum value of concentration k_0,

$$k_0 = \frac{k_j}{2} = 160 \text{ veh/km}$$

The slope of the line AB represents the velocity of the shock wave that is formed when the traffic is brought to a halt.

From the property of the parabola,

$$\frac{x^2}{(160)^2} = \frac{y}{2000} = \frac{800}{2000}$$

$$\therefore \quad x = \sqrt{\frac{800 \times 160 \times 160}{2000}} = 101.2$$

$$\therefore \quad k_A = 160 - 101.2 = 58.8$$

$$\therefore \quad \text{Slope of the line } AB = \frac{120}{320 - 58.8} = \frac{1200}{261.2} = 4.6$$

$\therefore$ Speed of the shock wave = 4.6 kmph

$\therefore$ Time in seconds to reach a distance of 75 m = $\frac{75 \times 3600}{4.6 \times 100}$ = 58.7 sec

Example-4.21 An observation on an Expressway yielded a count of 200 vehicles in a period of half an hour. Calculate the number of headways in this count:

(*i*) greater than 4.5 seconds and (*ii*) less than 18 seconds

Solution:

The mean time headway, $x = \bar{t} = \frac{T}{N} = \frac{1800}{200} = 9$ sec

$$P(h \geq i) = e^{-(t/\bar{t})}$$

$$P(h \geq 4.5) = e^{-(4.5/9)} = e^{-0.5} = 0.6065$$

Total number of headways counted = 200 – 1 = 199

Number of headways greater than 4.5 seconds = 199 × 0.6065 = 120

$$P(h \leq t) = 1 - e^{-(t/\bar{t})}$$

$$\therefore \quad P(h \leq 18) = 1 - e^{-(18/9)} = 1 - e^{-2}$$

$$= 1 - 0.1353 = 0.8647$$

$\therefore$ Number of headways less than 18 seconds

$= 199 \times 0.8647 = 172.07$

Example-4.22 A pre-timed four phase signal has critical lane flow rate for the first three phases as 200, 187 and 210 veh/hr with saturation flow rate of 1800 veh/hr/lane for all phases. The lost time is given as 4 seconds for each phase. If the cycle length is 60 seconds, find the effective green time of the fourth phase.

Solution:

Flow rates for the first three phase are given as:

q_1 = 200 veh/hr, q_2 = 187 veh/hr and q_3 = 210 veh/hr

Saturation flow rate is 1800 veh/hr/lane

Lost time, $L = 4 \times 4 = 16$ sec

Length of the cycle, $C_0 = 60$ sec

Now, $y_1 = \frac{q_1}{s_1} = \frac{200}{1800}$, $y_2 = \frac{q_2}{s_2} = \frac{187}{1800}$ and $y_3 = \frac{q_3}{s_3} = \frac{210}{1800}$

$$C_0 = \frac{1.5L + 5}{1 - y} \Rightarrow 60 = \frac{1.5 \times 16 + 5}{1 - y}$$

$$\Rightarrow \quad 60 = \frac{24 + 5}{1 - y} \Rightarrow y = 0.517$$

and $y = y_1 + y_2 + y_3 + y_4$

$$\Rightarrow \quad 0.517 = \frac{597}{1800} + y_4 \Rightarrow y_4 = 0.185$$

$$G = \frac{y_4}{y}(C_0 - L) = \frac{0.185}{0.517}(60 - 16) = 15.745 \text{ sec}$$

Summary

- Stability of vehicle and its safe movement on horizontal curve depends upon the width of vehicle and height of center of gravity.
- **Braking test** is conducted to measure the skid resistance of a pavement surface.
- Magnetic detectors, Pressure sensitive detectors and electronic detectors are pavement counting recorders.
- Annual average daily traffic is the average volume of traffic at a particular location over a full 365 days.
- **30th highest hourly volume** is the design hourly volume.
- When vehicles are moving with same speed then space mean speed is equal to time mean speed.
- **Floating car method** is used to carry out speed and delay study.
- **Origin and destination** study is used to plan the road network and other facilities for vehicular traffics.
- Traffic capacity is the ability of a roadway to accommodate traffic volume.
- Traffic signs are divided into regulatory signs, warning signs and informatory signs.
- Traffic signal is a device which is used to direct the traffic to stop and proceed at intersections using red and green traffic light signals automatically.
- **Green interval** is the actual duration for which green light of traffic is turned on.
- Methods of signal designing are Trial cycle method, Approximate method, Webster method and IRC method.
- Crossing conflicts are the major conflicts and merging and diverging conflicts are minor conflicts.

Important Expressions

1. Time mean speed, $V_t = \dfrac{\sum_{i=1}^{n} V_i}{n}$

2. Space mean speed, $V_s = \dfrac{3.6\,dn}{\sum_{i=1}^{n} t_i}$

3. Average journey time, $\bar{t} = t_w - \dfrac{n_y}{q}$, $q = \dfrac{n_a + n_y}{t_a + t_w}$

4. Poisson distribution, $P(n) = \dfrac{(\lambda t)^n e^{-\lambda t}}{n!}$

5. Optimum cycle time, $C_0 = \dfrac{1.5L+5}{1-Y}$

6. Rotary capacity, $Q = \dfrac{280w\left(1+\frac{e}{w}\right)\left(1-\frac{p}{3}\right)}{1+\frac{w}{l}}$

Objective Brain Teasers

Q.1 An intersection approach has an approach flow rate of 1000 vph, a saturation flow rate of 2800 vph, a cycle length of 40 seconds and effective green ratio for the approach is 0.55. What is the critical capacity of the lane.
(a) 1540 veh/hr (b) 1600 veh/hr
(c) 1660 veh/hr (d) 2000 veh/hr

Q.2 How many number of points of conflicts can rise with one-way regulation in both directions on an intersection having 4 legs?
(a) 4 (b) 6
(c) 8 (d) 10

Q.3 Free flow speed is 50 kmph, jam density is 70 veh/km. At a particular condition it is observed that there are 20 vehicles on a lane in 1 km length of road. The average speed on the road is
(a) 45 kmph (b) 35.5 kmph
(c) 37.5 kmph (d) 40.0 kmph

Q.4 For designing a 2-phase fixed type signal at an intersectin having north-south and east-west road where only straight ahead traffic is permitted, the following data is available. The time lost per cycle is 12 seconds, the cycle length as per webster's approach is

Parameter	North	South	East	West
Design hour flow(PCU/hr)	1000	700	900	550
Saturation (PCU/hr)	2500	2500	3000	3000

(a) 67 sec (b) 77 sec
(c) 87 sec (d) 91 sec

Q.5 The length of the side of warning sign boards of roads is
(a) 30 cm (b) 40 cm
(c) 45 cm (d) 50 cm

Q.6 Match **List-I** with **List-II** and select the correct answers by using the codes given below the lists:

List I
A. Economic studies
B. Financial studies
C. Traffic studies
D. Engineering studes

List II
1. For road location and alignment
2. For population and agricultural pattern
3. For ascertaining the source of income
4. For traffic volume and traffic flow patterns

Codes:

	A	B	C	D
(a)	1	2	3	4
(b)	2	3	4	1
(c)	3	4	2	1
(d)	1	3	2	4

Q.7 If the normal traffic growth is 50%, the generated traffic is 20%, the development traffic is 40%, according to an origin and destination survey on a section of state Highway in 1995 is 10,000, the future ADT (2000) is
(a) 11050 (b) 15050
(c) 21100 (d) None of these

Q.8 Match **List-I** with **List-II** and select a suitable answer by using the codes given below the lists.

List-I
A. Right of way
B. Carriage way
C. Shoulders
D. Formation width

List-II
1. The portions of the highway between the outer edge of the pavement and inside edge of slope.
2. The total width on which a formation is raised.
3. Width of the highway between the boundary lines of the property abutting it.
4. The cambered surface

Codes:

	A	B	C	D
(a)	1	2	3	4
(b)	3	4	1	2
(c)	2	3	4	1
(d)	4	1	2	3

Q.9 Two major roads with two lanes each are crossing in an urban area to form an un-controlled intersection. The number of conflict points when both roads are one way is "*X*" and when one road is one way and other one is two way is "*Y*". The ratio of *X* to Y is

(a) 0.33 (b) 0.25
(c) 0.545 (d) 0.73

Q.10 The percentage composition of single lane Ambedkar Road is

4 wheeler = 80
2 wheeler = 10
6 wheeler = 2
10 wheeler = 8

The average space occupied by each type of vehicle when stationary in jammed condition is

4 wheeler = 7 m
2 wheeler = 4 m
6 wheeler = 10 m
10 wheeler = 12 m

It has been observed that maximum free speed of traffic is 60 kmph. What will be the Q_{max} in vehicle/hour/lane

(a) 1800 (b) 1476
(c) 1575 (d) 1600

Q.11 Which interchange is used when two high volume and high speed facilities intersect each other?

(a) Cloverleaf interchange
(b) Diamond interchange
(c) T-interchagne
(d) Half cloverleaf interchange

Q.12 As per IRC guideline, for a rotary design speed of 40 kmph, the minimum radius of horizontal curve, when superelevation is zero, is [Assume coefficient of friction, $f = 0.45$]

(a) 14 m (b) 23 m
(c) 28 m (d) 32 m

Q.13 The free speed of cars, having length of 5 m, on a highway is 75 kmph. The maximum capacity (in cars/hour/lane) of a lane of road is

(a) 3750 (b) 3500
(c) 3200 (d) 2950

Q.14 A fixed time four-phase traffic signal is designed by Webster's method. The critical flow ratio for four phase are 0.15, 0.20, 0.25 and 0.20 respectively, and lost time for each phase is 3 seconds. The optimum cycle length (in seconds) is

(a) 115 (b) 120
(c) 230 (d) 240

Q.15 What is the theoretical capacity (in vehicle/hour/lane) of a traffic lane with one way traffic flow at a stream speed of 50 kmph?
[Assume the average space gap between vehicles to follow the relation $S_g = 0.278$ V.t. where '*V*' is the stream speed in kmph, '*t*' is the average reaction time = 0.7 sec, average length of vehicles = 5.0 m]

(a) 3125 (b) 3230
(c) 3330 (d) 3395

Q.16 The spot speed measurements (in km per hour) at a particular site were as given below:
46, 56, 40, 92, 63, 75, 68, 90, 70, 65
The difference between time mean speed and the space mean speed is

(a) 2.90 kmph (b) 3.40 kmph
(c) 4.10 kmph (d) 4.80 kmph

Q.17 The free speed of a car on a 4-lane road is 80 km per hour, while that of a truck is 40 km per hour. The length of the car is 5 m, whereas the length of the truck is 10 m. The PCU of the truck is

(a) 2 (b) 4
(c) 6 (d) 8

Q.18 On a right angled road intersection with two way traffic on both the roads, the total number of conflict points are

(a) 6 (b) 11
(c) 18 (d) 24

Q.19 Which one of the statement is INCORRECT in design of hourly traffic volume?

(a) Design traffic volume is determined from the plot between hourly volume and the number of hours in a year that the traffic volume is exceeded.
(b) Thirtieth highest hourly traffic volume is the hourly volume that will be exceeded only 30 times in a year and all other hourly volumes of the year will be less than this value.
(c) The highest or peak hourly volume of the year will be too high and it will not be economical to design the facilities according to this volume.
(d) The annual average hourly volume found from AADT will not at all be sufficient, because during considerable period of the year there will be severe congestion.

Q.20 In a rotary, the proportion of weaving traffic is 0.60 and width of weaving section is 15 m. The length of the weaving section between channelizing ends is 75 m and the average width of entry and non-weaving section is 5 m. The capacity (in PCU/hr) of rotary is

(a) 3733 (b) 3453
(c) 2968 (d) 2551

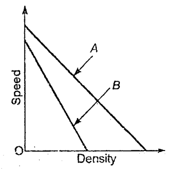
Q.21 The average normal flow of traffic on cross roads *A* and *B* during design period are 500 and 300 PCU per hour; the saturation flow values on these roads are estimated as 1500 and 1000 PCU per hour respectively. The all red time required for pedestrian crossing is 12 sec. What is the optimum cycle time (in sec). for two phase traffic signal with pedestrian crossing?

(a) 52 (b) 69
(c) 79 (d) 83

Q.22 Which interchange is used when two high volume and high speed facilities intersect each other?

(a) Cloverleaf interchange
(b) Diamond interchange
(c) T-interchange
(d) Half cloverleaf interchange

Q.23 In the given figure, the speed-density relationship on two different roads with varying lane (road) widths are plotted.



(a) Road *B* has wider lanes than *A*.
(b) Road *A* has wider lanes than *B*.
(c) Both roads *A* and *B* have same lane width.
(d) Nothing can be infered from figure.

Q.24 A vehicle of weight 2.0 ton skids through a distance equal to 40 m before colliding with another parked vehicle of weight 1.0 ton. After collision, both the vehicles skid through a distance equal to 12 m before stopping. Assuming coefficient of friction as 0.5, the initial speed of the moving vehicle is

(a) 75.60 kmph (b) 81.23 kmph
(c) 87.68 kmph (d) 92.29 kmph

Q.25 The average normal flow of traffic on cross roads P and Q during design period are 400 and 250 pcu per hour respectively; the saturations flow values on these roads are estimated as 1250 and 1000 pcu per hour respectively. The all-red time required for pedestrian crossing is 12 seconds. Assume amber time of 2.0 seconds each for clearance. Total cycle time for two phase traffic signal as per Webster's method is

(a) 67.44 second (b) 59.33 second
(c) 70.45 second (d) 77.45 second

Q.26 Traffic flow in an urban area at right angled intersection of two major roads in the design year are given below:

Approach road	Traffic Flow in PCU/hr		
	Left turning	Straight ahead	Right turning
North	415	643	350
East	408	450	402
South	549	358	424
West	450	423	493

What is the maximum proportion of weaving traffic to total traffic?

(a) 0.78 (b) 0.64
(c) 0.52 (d) 0.72

Q.27 A student riding a bicycle on a 5 km one-way street takes 40 minutes to reach home. The student stopped for 15 minutes during this ride. 60 vehicles overtook the student (assume the number of vehicles overtaken by the student is zero) during the ride and 45 vehicles while the student stopped. The speed of vehicle stream on that road (in km/hr) is

(a) 7.5 (b) 12
(c) 40 (d) 60

Answers

1. (a)	2. (b)	3. (b)	4. (b)	5. (c)
6. (b)	7. (c)	8. (b)	9. (c)	10. (a)
11. (a)	12. (c)	13. (a)	14. (a)	15. (d)
16. (c)	17. (b)	18. (d)	19. (b)	20. (a)
21. (c)	22. (a)	23. (b)	24. (d)	25. (a)
26. (d)	27. (d)			

Conventional Practice Questions

Q.1 A vehicle travelling at 40 kmph was stopped within 1.8 seconds after the applications of the brakes. Determine the average skid resistance.

[Ans. 0.63]

Q.2 A fixed time 2-phase signal is to be provided at an intersection having a North-South and an East-West road where only straight ahead traffic is permitted. The design hour flows from the various arms and the saturation flows for these arms are given in the following table:

Calculate the optimum cycle time and green times for the minimum overall delay. The intergreen time should be the minimum necessary for efficient operation. The time lost per phase due to starting delays can be assumed to be 2 seconds. The value of the amber period is 2 seconds. Sketch the timing diagram for each phase.

	North	South	East	West
Design hour flow in PCU/hour	800	400	750	1000
Saturation flow in PCU/hour	2400	2000	3000	3000

[Ans. 40 sec, 18 sec and 14 sec]

Q.3 One lane of a 2 lane one way carriageway is closed for repairs. The maximum mean free speed under conditions of low flow in the 2 lane portion is 65 kmph. Under conditions of low flow, observations show that the maximum mean free speed in the bottleneck is also 65 kmph. The average space headway when vehicles are stationary is 12.5 m. The volume of traffic on the 2 lane road is 1800 veh/hour. Assuming that the speed concentration relation is linear, find:

(*i*) the mean speed of traffic through the bottleneck caused by the closure of one lane.

(*ii*) the mean speed of traffic in the congested conditions immediately on the approach to the bottleneck.

(*iii*) the mean speed of traffic on the carriageway clear of the influence of the bottleneck.

Ans. (*i*) 32.5 kmph
(*ii*) 95.6 kmph
(*iii*) 50.7 kmph

Q.4 An observer counts 240 veh/h at a specific highway location. Assume that the vehicle arrival at the location is Poisson distributed, find the probability of having one vehicle arriving over a 30-second time interval.

[Ans. 0.27]