51 State the factors that govern the length of summit curve. How is it decided ?

52 Discuss the requirement of summit curves and its shape.

53 Explain how the vertical curves on a hump formed due to the presence of a culvert slightly above the profile may be designed.

54 Explain the factors based on which the length of valley curve is designed.

55 Discuss the problems in highway valley curves and the best shape of a valley curve.

56 A vertical summit curve is formed when an ascending gradient of 1 in 25 m another ascending gradient of 1 in 100. Find the length of the summit curve to provide the required stopping sight distance for a design speed of 80 kmph.

57 The deviation angle at a summit curve is 0.05 and the overtaking sight distance is 300 m. Find the length of summit curve required.

58 An ascending gradient of 1 in 50 meets a descending gradient of 1 in 80. Determine the length of summit curve to provide (a) ISD (b) OSD, for design speed of 80 kmph. Assume all other data.

59 A valley curve is formed by a descending gradient of 1 in 40 which meets an ascending gradient of 1 in 30.

(i) Design the total length of valley curve if the design speed is 100 kmph so as to fulfil both comfort condition and head light sight distance for night driving, after calculating the SSD required.

(ii) Find the position of the lowest point of the valley curve to locate a culvert.

Chapter 5
Traffic Engineering

5.1 INTRODUCTION

5.1.1 General

Traffic engineering is that branch of engineering which deals with the improvement of traffic performance of road networks and terminals. This is achieved by systematic traffic studies. Scientific analysis and engineering applications. The method includes planning and geometric design on one hand and regulation and control on the other. Traffic Engineering therefore deals with the application of scientific principles, tools, techniques and findings for safe, rapid, convenient and economic movement of people and goods.

The road traffic is composed of various categories of vehicular traffic and the pedestrian traffic. Each category of vehicular traffic has two components, the human element as the driver and his machine as vehicles. Traffic engineering has also to be recognized and governed by social and physical science. The profession of traffic engineering as known today has evolved with the advent of motor vehicle. During the last few decades significant advances have been made in many phases of the profession. Advanced study and training facilities have been made available at several universities and institution notably in the U.S.A. Traffic engineering has now been recognized as an essential tool in the improvement of traffic operations in metropolitan cities like Bombay, Delhi, Calcutta and Madras.

Definition

Institute of Traffic Engineers, U.S.A. defines, "Traffic engineering is that phase of engineering which deals with planning and geometric design of streets, highways, abutting lands, and with traffic operation thereon, as their use is related to the safe, convenient and economic transportation of persons and goods".

Professor Ress Blunden of California University has proposed a modified definition, "Traffic engineering is the science of measuring traffic and travel, the study of the basic laws relating to traffic flow and generation and application of this knowledge to the professional practice of planning, designing and operating traffic systems to achieve safe and efficient movement of persons and goods".

5.1.2 Scope of Traffic Engineering

The basic object of traffic engineering is to achieve efficient free and rapid flow of traffic, with least number of traffic accidents. Factual studies of traffic operations provide the foundation for developing methods for improvement in general and for solving specific problems.

The study of traffic engineering may be divided into six major sections, viz.:

(i) Traffic characteristics

(ii) Traffic studies and analysis

(iii) Traffic operation-control and regulation

(iv) Planning and analysis

(v) Geometric design

(vi) Administration and management

Study of traffic characteristics is the most essential prerequisite for any improvement of traffic facilities. The traffic characteristics are quite complex with various types of road users in the roads moving with different motives. The human psychology is to be given particular attention. The study of vehicular characteristics is an essential part. Apart from these the various studies to be carried out on the actual traffic include speed, volume, capacity, travel patterns, origin and destination, traffic flow characteristics; parking and accident studies.

Various aspects that are covered under traffic operations are regulations, control and the warrants for application of controls. Regulations may be in the form of laws and ordinances or other traffic regulatory measures such as speed limit etc. Installation of traffic control devices like signs, signals and islands are most common means to regulate and control the traffic. Actual adoption of traffic management measures, such as traffic regulations and control need adequate attention.

Traffic planning is a separate phase for major highways like *express-ways*, arterial roads, mass transit facilities, and parking facilities. All the aspects such as cross section and surface details, sight distance requirement, horizontal and vertical alignment, manoeuvre areas and intersections and parking facilities are to be suitably designed for better performance.

The various phases of traffic engineering are implemented with the help of *Engineering, Enforcement and Education* or "3-Es". Enforcement is usually made through traffic laws, regulations and control. Education may be possible by sufficient publicity and through schools and television. It aims at improving the human factor in traffic performance. Engineering phase is the one which is constructive. It deals with improvement of road geometrics, providing additional road facilities and installation of suitably designed traffic control devices.

5.2 TRAFFIC CHARACTERISTICS

5.2.1 Road User Characteristics

The human element is involved in all actions of the road users either as pedestrian, cyclist, cart driver or motorist. The physical, mental and emotional characteristics of

human beings affect their ability to operate motor vehicle safely or to service as a pedestrian. Hence it is important to the traffic engineer to study the characteristics and limitations of the road users.

The various factors which affect road user characteristics may broadly be classified under four heads :

Physical

Mental

Psychological and

Environmental

Physical characteristics

The physical characteristics of the road users may be either permanent or temporary. The pavement characteristics are the vision, hearing, strength and the general reaction to traffic situations.

Vision plays the most important role of all these. These include the acuity of vision, peripheral vision and eye movement; glare vision, glare recovery and depth judgement. Minimum standards for acuity of vision are often laid down by licensing authorities. Field of clearest and acute vision is within a cone whose angle is only 3 degrees, though the vision is fairly satisfactory up to 10° in general and even upto 20° in the horizontal plane. However in the vertical plane the field of clear vision may be about two thirds of that in the horizontal plane. These factors are particularly taken care of while designing and installing control devices. As the field of clear vision is limited, the road users have to often shift their eyes within the peripheral field to obtain clear vision. The total time taken for the eye movement depends on some of the physical characteristics including the response to stimuli. The effects of glare, adoptability to changes of light i.e., darkness to light and bright light to darkness, should also be studied. The depth judgement is important for a driver in judging distance and speed of vehicles and other objects ahead.

Hearing helps drivers in a way, though it is more important for pedestrians and cyclists. Though strength is not an important factor in general, lack of strength may make parking maneouvers difficult, particularly for heavy vehicles. The reaction to traffic situations depends on the time required to perceive and understand the traffic situation and to take the appropriate action. This depends on many factors such as permanent and temporary physical factors mental and psychological set up, speed and environmental factors. Also the time required to take an appropriate action depends on the type of the problem and the familiarity. The PIEV theory explaining the total reaction time has already been discussed in Art. 4.3.2.

The temporary physical characteristics of the road users affecting their efficiency are fatigue, alcohol or drugs and illness. All these reduce alertness and increase the reaction time and also affect the quality of judgement in some situations.

Mental characteristics

Knowledge, skill, intelligence experience and literacy can affect the road user characteristics. Knowledge of vehicle characteristics, traffic behaviour, driving practice, rules of roads and psychology of road users will be quite useful for safe traffic operations. Reactions to certain traffic situations become more spontaneous with experience. Understanding the traffic regulation and special instruction and timely action depends on intelligence and literacy.

Psychological factors

These affect reaction to traffic situations of road users to a great extent. The emotional factors such as attentiveness, fear anger, superstition impatience, general attitude towards traffic and regulations and maturity also come under this. Distractions by non-traffic events and worries reduce attentiveness to traffic situations. Dangerous actions are likely due to impatience. Some road users do not pay due regard to the traffic regulations and do not have the right attitude towards the traffic.

Environmental factors

The various environmental conditions affecting the behaviour of road user are traffic stream characteristics, facilities to the traffic, atmospheric conditions and the locality. The traffic stream may consist of mixed traffic or heavy traffic whereas the facilities to overtake for faster vehicles may be limited. The adoptability to different traffic stream characteristics depends on the driver's characteristics as well as the motivation. The purpose of entering the traffic stream can be social, recreational, business, routine movement or an emergency dash. The time, place and route are fundamentally chosen by the road user based on the needs. Whatever be the motive of movement, once the individual enters the traffic stream, the road user, is usually motivated by the desire for time-distance economy on one hand, and comfort and safety on the other. Together with modifying factors of motivation, there is a great variation among road users and their behaviour in every traffic stream. The locality may be a shopping centre or a place with other distractions to the road users, thus affecting their behaviour. The other environmental factors of importance are the weather visibility and other atmospheric conditions.

The total reaction time or the "PIEV" time of the drivers vary considerably from driver to driver based on the above road users characteristics. But the reaction time of a particular driver may vary depending on the type of the problem and also environmental and modifying factors.

Driver characteristics such as the simple, reaction time, depth judgement, field of vision, visual acuity, glare recovery etc. may be measured in the laboratory using Driver Testing Equipment.

5.2.2 Vehicular Characteristics

It is quite important to study the various vehicular characteristics which affect the design and traffic performance, because it is possible to design a road for any vehicle but not for an indefinite vehicle. The basic criterion of highway engineering is to cater for the needs of existing and anticipated traffic. It will not be economically feasible to keep on increasing the geometric standards and thickness of pavements from time to time to meet the needs of a few vehicles whose dimensions and weight are increased. Hence the vehicle standards should be uniform at least within a country, keeping in view the large percentage of existing vehicles and those likely to be manufactured in the near future. The standards for the dimensions and weights of vehicles should be consistent with the road facilities now available or could be made available in the near future. The various vehicular characteristics affecting the road design may be classified as static and dynamic characteristic of the vehicles.

Static characteristics of vehicles affecting road design are the dimensions, weight and maximum turning angle. The height of vehicle affects the clearance of the overhead structures. The height of driver seat affects the visibility distance and the height of head

light affects the head light sight distance at valley curves. The field of vision ahead for the driver also depends on the design of wind shield and the front portion of the vehicle body. The clearance below the chassis, approach, departure and ramp angles of the vehicle affects the design of verticle profile of drive ways, humps and dips. The length of vehicles affects the capacity, overtaking distance and maneuverability of vehicles. The minimum turning radius depends on the length of wheel base and the features of the steering system and this affects design of sharp curves for the manoeuvre of vehicles at slow speeds. Gross weight, axle and wheel loads of vehicle govern the structural design of pavements and cross drainage structures.

Dynamic characteristics of vehicles affecting road design are speed, acceleration and braking characteristics and some aspects of vehicle body design. The speed and acceleration depends upon the power of the engine and the resistances to be overcome and are important in all the geometric design elements. The deceleration and braking characteristics guide safe vehicle operation. The stability of vehicle and its safe movement on horizontal curves are affected by the width of wheel base and the height of centre of gravity. The riding comfort on vertical curves depends on the design of suspension system of the vehicle. The impact characteristics on collision and the injuries to the occupants depends on the design of the bumper and body of vehicle. Some of the vehicle characteristics have been discussed below in detail.

Vehicle dimensions

The dimensions to be mainly considered are the overall width, height, and length of different vehicles, particularly of the largest ones. The width of the vehicle affects the width of the traffic lanes, shoulders and parking facilities. If the width of the lanes are not adequate in view of the widest vehicles using the roads, the capacity of road will decrease. Height of the vehicle affects the clearance to be provided under structures such as overbridges underbridges, electric and other service lines. Length of the vehicle is an important factor in the design of horizontal alignment as it effects the extra width of pavement and minimum turning radius. Length affects the safe overtaking distance, capacity of a road and parking facilities. The length should also be considered in the design of valley curves and dips. The maximum allowable width, height and length of vehicles have been standardized by the Indian Roads Congress and are as given in Table 5.1 (a). The configurations of different types of transport vehicles and their axle arrangement are given in Fig. 5.1.

Table 5.1 (a) Maximum Dimensions of Road Vehicles

Dim. of Vehicle	Details	Maximum dim., m (excluding front & rear bumpers)
Width	All vehicles	2.50
Height	(a) Single-decked vehicle for normal application	3.80
	(b) Double-decked vehicle	4.75
Length	(a) Single-unit truck with two or more axles (types 2, 3)	11.00
	(b) Single-unit bus with two or more axles (types 2, 3)	12.00
	(c) Semi-trailer tractor combinations (Types 2-S1, 2-S2, 3-S1, 3-S2)	16.00
	(d) Tractor and trailer combinations (Types 2-2, 3-2, 2-3, 3-3)	18.00

No combination is allowed to be of more than two units, and no such combination, laden or unladen is allowed to have an overall length exceeding 18 m. The other dimensions to be considered are the wheel base (which affects the minimum turning

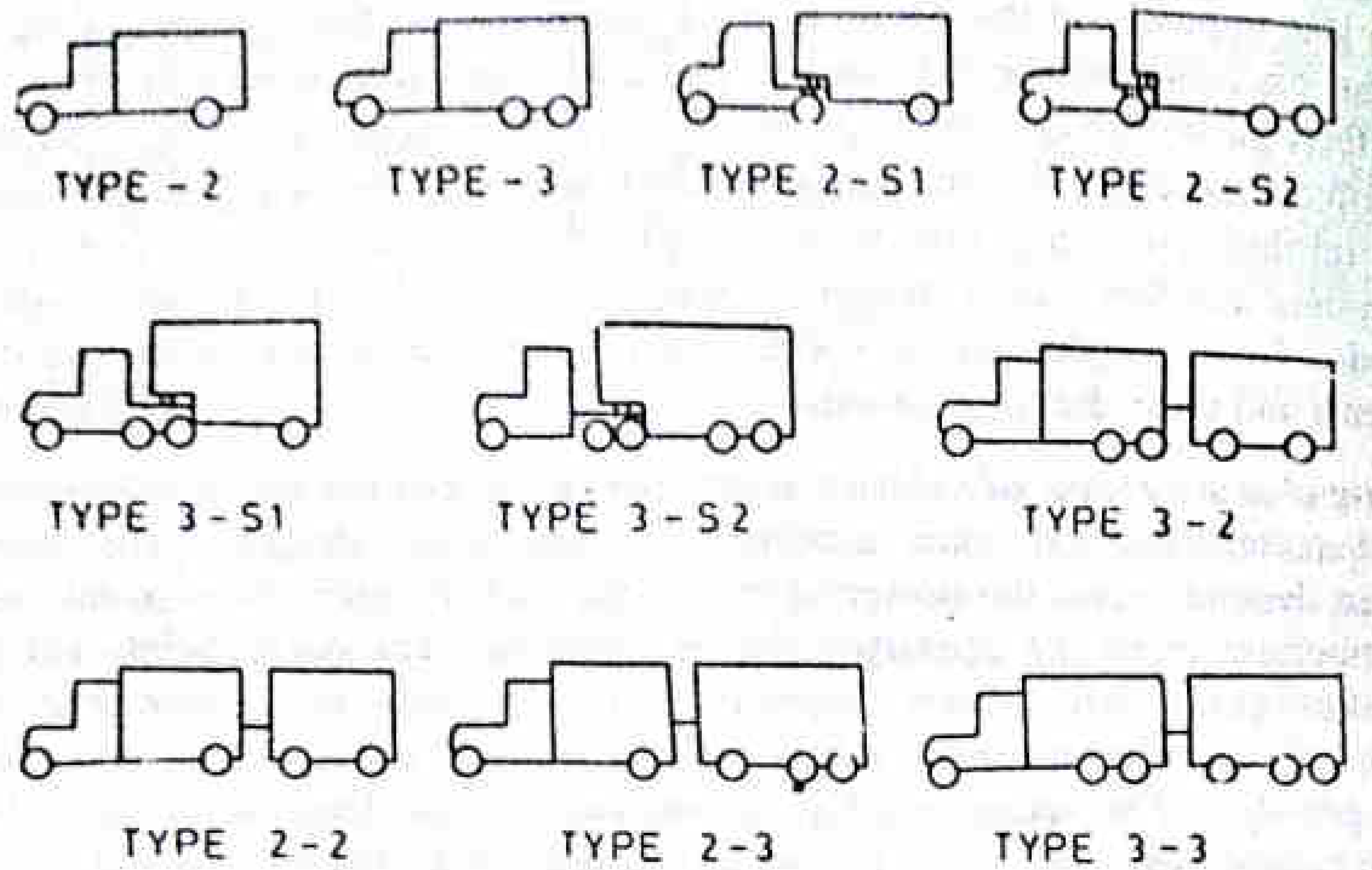


Fig. 5.1 Types of Road Transport Vehicles

radius and extra widening required) the front, rear and centre clearances and the approach, departure and ramp angles (which affect the design of vertical profile for highways and driveways).

Weight of loaded vehicle

The maximum weight of loaded vehicle affects the design of pavement thickness and gradients. In fact the limiting gradients are governed by both the weight and power of the heavy vehicles. The maximum permissible gross weights and axle weights have been standardized by IRC as per Table 5.1 (b).

Table 5.1 (b) Maximum Permissible Gross Weight and Axle Weight of Transport Vehicles

Vehicle type	Maximum Gross weight, tonnes	Maximum axle weight, tonnes			
		Truck/Tractor		Trailer	
		FAW	RAW	FAW	RAW
Type 2 (Both axles single tyre)	12.0	6	6.0		
Type 2 : (FA-Single tyre RA-Dual tyre)	16.0	6	10.2		
Type 3	24.0	6	18 (TA)		
Type 2-S1	26.4	6	10.2		10.2
Type 2-S2	34.2	6	10.2		18 (TA)
Type 3-S1	34.2	9	18 (TA)		10.2
Type 3-S2	42.0	6	18 (TA)		18 (TA)
Type 2-2	36.6	6	10.2	10.2	10.2
Type 3-2	44.4	6	18 (TA)	10.2	10.2
Type 2-3	44.4	6	10.2	10.2	18 (TA)
Type 3-3	52.2	6	18 (TA)	10.2	18 (TA)

FAW = Weight on Front Axle; RAW = Weight on Rear Axle

TA = Tandem axle, fitted with 8 tyres.

Power of vehicle

The power of the heaviest vehicles and their loaded weights govern the permissible and limiting values of gradient on roads. In this regard the total resistances to traction consisting of inertia, rolling resistance, air resistance and grade resistance are considered. From the total hauling capacity and the power required to overcome the total tractive resistance, it is possible to determine the speed and acceleration of the vehicle which in turn is useful in traffic regulation, planning, and design.

Speed of vehicle

The vehicle speed affects, (i) sight distances (ii) superelevation, length of transition curve and limiting radius on horizontal curves (iii) length of transition curves on vertical valley curves and on humps (iv) width of pavement and shoulders on straight and on horizontal curves (v) design gradient (vi) capacity of traffic lane (vii) design and control measures on intersections.

Thus the design speed controls most of the geometric features of highways.

Braking characteristics

The deceleration and braking characteristics of vehicles depend on the design and type of braking system (such as mechanical, fluid or air brake) and its efficiency. The safety of vehicle operation, stopping distance and the spacing between the two consecutive vehicles in a traffic stream are affected by the braking capacity. Thus the highway capacity and overtaking sight distance requirements also indirectly get affected.

Braking Test

It is possible to measure the skid resistance of pavement surface under the prevailing conditions by conducting braking tests on the road at the desired running speed. If the brakes are applied till the vehicle comes to stop, it may be assumed that wheels are fully locked and the brake efficiency is 100 percent. Atleast two of the following three measurements are needed during the braking tests in order to determine the skid resistance of the pavement :

(i) Braking distance, L metre

(ii) Initial speed, u m/sec.

(iii) Actual duration of brake application, t second

The method of calculating the average skid resistance of the pavement using two of the above three values has been illustrated with the help of Examples 5.1, 5.2 and 5.3.

Example 5.1

In a braking test, a vehicle traveling at a speed of 30 kmph was stopped by applying brakes fully and the skid marks were 5.8 m in length. Determine the average skid resistance of the pavement surface.

Solution

$$\text{Initial speed } u = \frac{30}{3.6} = 8.33 \text{ m/sec}$$

$$\text{Braking distance } L = 5.8 \text{ m} = \frac{u^2}{2gf} \quad \text{(see Eq. 4.1)}$$

$$\text{Average skid } f = \frac{8.33^2}{2\times 9.8\times 5.8} = 0.61$$

Example 5.2

A vehicle travelling at 40 kmph was stopped within 1.8 seconds after the application of the brakes. Determine the average skid resistance.

Solution

Initial speed $u = \frac{40}{3.6} = 11.11$ m/sec; Braking time t = 1.8 sec. Using the fundamental relation of motion for uniform acceleration/retardation,

$$v = u + at,\ v = 0,\ \text{retardation } a = \frac{u}{t} = \frac{11.11}{1.8} = 6.17 \text{ m/sec}^2$$

$$\text{From the relation, force } F = m\,a,\ W f = \frac{W a}{g}$$

$$\text{Average skid resistance } f = \frac{a}{g} = \frac{6.17}{9.88} = 0.63$$

Example 5.3

A vehicle was stopped in 1.4 second by fully jamming the brakes and the skid marks measured 7.0 m. Determine the average skid resistance.

Solution

Using the fundamental relations of motion for uniform acceleration/retardation

(i) $v = u + at$, as the final velocity $v = 0$, $u = -at$

(ii) $v^2 - u^2 = 2\,a\,s$

$$s = -\frac{u^2}{2a} = \frac{a^2 t^2}{2a} \text{ and therefore } a = \frac{2s}{t^2}$$

Given braking distance L = 7.0 m = s

and braking time t = 1.4 sec

$$\text{Average skid resistance } f = \frac{a}{g} = \frac{2\times 7.0}{9.8\times 1.4^2} = 0.729$$

Example 5.4

A vehicle moving at 40 kmph speed was stopped by applying the brake and the length of skid mark was 12.2 m. If the average skid resistance of the pavement is known to be 0.70, determine the brake efficiency of the test vehicle.

Solution

$$v = \frac{40}{3.6} = 11.11 \text{ m/sec, } L = 12.2 \text{ m, } f = 0.70$$

Average skid resistance developed

$$f' = \frac{v^2}{2\,gL} = \frac{11.11^2}{2\times 9.8\times 12.2} = 0.516$$

$$\text{Brake efficiency, \%} = \frac{100 f'}{f} = \frac{100\times 0.516}{0.70} = 73.7\%$$

Off Tracking

When a four or six wheeled vehicle, such as car or bus (vehicle other than two and three wheelers) negotiates a horizontal curve at relatively slow speed, the rear wheels do not race the same path as the corresponding front wheels, as explained in Art. 4.4.6 under 'Mechanical Widening'. At relatively slow speeds when the centrifugal force developed is lesser than the counteracting forces due to the superelevation and transverse friction, the rear wheels follow paths on the inner side of the horizontal curve as compared with the path followed by the corresponding set of front wheels. This difference in distance between the curved wheel paths of a particular set of front and rear wheels (i.e., either the set of front and rear wheels on the outer side of the horizontal curve or the set on the inner side) is called off-tracking or the mechanical widening for a vehicle which is equal to $l^2/2R$ (see Eq. 4.15). Thus the off tracking depends on two factors :

(i) the length of wheel base or the distance between the front and real axles of the vehicle and

(ii) the turning angle or the mean radius of the horizontal curve traversed.

Example 5.5

A vehicle has a wheel base of 6.5 m. What is the off tracking while negotiating a curved path with a mean radius 32 m.

Solution

$$l = 6.5 \text{ m};\ R = 32 \text{ m}$$

$$\text{Off tracking} = \frac{l^2}{2R} = \frac{6.5^2}{2\times 32} = 0.66 \text{ m}$$

5.2.3 Traffic Studies

Traffic studies or surveys are carried out to analyse the traffic characteristics. These studies help in deciding the geometric design feature and traffic control for safe and efficient traffic movements. The traffic surveys for collecting traffic data are also called *traffic census*.

The various traffic studies generally carried out are :

(a) Traffic volume study

(b) Speed studies

(i) spot speed study

(ii) speed and delay study

(c) Origin and destination (O & D) study

(d) Traffic flow characteristics

(e) Traffic capacity study

(f) Parking study

(g) Accident studies or the traffic flop

Traffic volume study

Traffic volume is the number of vehicles crossing a section of road per unit time at any selected period. Traffic volume is used as a quantity measure of flow; the commonly used units are vehicles per day and vehicles per hour. A complete traffic volume study may include the classified volume study by recording the volume of various types and classes of traffic, the distribution by direction and turning movements and the distribution on different lanes per unit time. The objects and uses of traffic volume studies are given below :

(a) Traffic volume is generally accepted as a true measure of the relative importance of roads and in deciding the priority for improvement and expansion.

(b) Traffic volume study is used in planning, traffic operation and control of existing facilities and also for planning and designing the new facilities.

(c) This study is used in the analysis of traffic patterns and trends.

(d) Classified volume study is useful in structural design of pavements, in geometric design and in computing roadway capacity.

(e) Volume distribution study is used in planning one-way streets and other regulatory measures.

(f) Turning movement study is used in the design of intersections, in planning signal timings, channelization and other control devices.

(g) Pedestrian traffic volume study is used for planning side walks, cross walks subways and pedestrian signals.

There are variations in traffic flow from time to time. Hourly traffic volume varies considerably during a day, the peak hourly volume may be much higher than average hourly volume. Daily traffic volumes vary considerably in a week and there are variations with season. Hence if a true picture is to be obtained, the hourly traffic volume should be known along with the patterns of hourly, daily and seasonal variations. In classified traffic volume study, the traffic is classified and the volume of each class of traffic viz., buses, truck, passenger-cars, other light vehicles, rickshaws, tongas, bullock carts, cycles and pedestrians is found separately. The direction of each class of traffic flow is also noted. At intersections the traffic flow in each direction of flow including turning movements are recorded.

Counting of traffic volume

Traffic volume counts may be done by mechanical counters or manually.

Mechanical counters

These may be either fixed (permanent) type or portable type. The mechanical counter can automatically record the total number of vehicles crossing a section of the road in a desired period. The working may be by the effect of impulses or stimuli caused by traffic movements on a pneumatic hose placed across the roadway or by using any other type of sensor. Traffic count is recorded by electrically operated counters and recorders capable of recording the impulses. The impulses caused by vehicles of light weight may not be enough in some cases to actuate the counter. Also it is not possible to easily record pedestrian traffic by this method. Other methods of working the mechanical detectors are by *photo-electric cells, magnetic detector* and *radar detectors*. The main advantage of mechanical counter is that it can work throughout the day and night for the desired period, recording the total hourly volume, which may not be practicable in manual counting. The main drawback of the mechanical counter is that it is not possible to get the traffic volumes of various classes of traffic in the stream and the details of turning movements.

Manual counts

This method employs a field team to record traffic volume on the prescribed record sheets. By this method it is possible to obtain data which can not be collected by mechanical counters, such as vehicle classification, turning movements and counts where the loading conditions or number of occupants are required. However it is not practicable to have manual count for all the 24 hours of the day and on all days round the year. Hence it is necessary to resort to statistical sampling techniques in order to cut down the manual hours involved in taking complete counts, First the fluctuations of traffic volume during the hours of the day and the daily variations are observed. Then by selecting typical short count periods, the traffic volume study is made by manual counting. Then by statistical analysis the peak hourly traffic volumes as well as the average daily traffic volumes are calculated. This method is very commonly adopted due to the specific advantages over other methods.

Presentation of traffic volume data

The data colleted during the traffic volume studies are sorted out and are presented in any of the following forms depending upon the requirements.

(a) *Annual average daily traffic* (AADT or ADT) of the total traffic as well as classified traffic are calculated. This helps in deciding the relative importance of a route and in phasing the road development programme. In order to convert the different vehicle classes to one class such as passenger car, conversion factors known as *Passenger Car Units* (PCU) are used. (see Tables 5.2 and 5.3).

(b) Trend charts showing volume trends over period of years are prepared. These data are useful for planning future expansion, design and regulation.

(c) Variation charts showing hourly, daily and seasonal variations are also prepared. These help in deciding the facilities and regulation needed during peak traffic periods.

(d) Traffic flow maps along the routes, (the thickness of the lines representing the traffic volume to any desired scale), are drawn. These help to find the traffic volume distribution at a glance.

(e) Volume flow diagram at intersections either drawn to a certain scale or indicating traffic volume as shown in Fig. 5.2 are prepared, thus showing the details of crossing and turning traffic. These data are needed for intersection design.

(f) *Thirtieth highest hourly volume* or the design hourly volume is found from the plot between hourly volume and the number of hours in an year that the traffic volume is exceeded. See Fig. 5.3. The 30^{th} highest hourly volume is the hourly volume that will be exceeded only 29 times in a year and all other hourly volumes of the year will be less

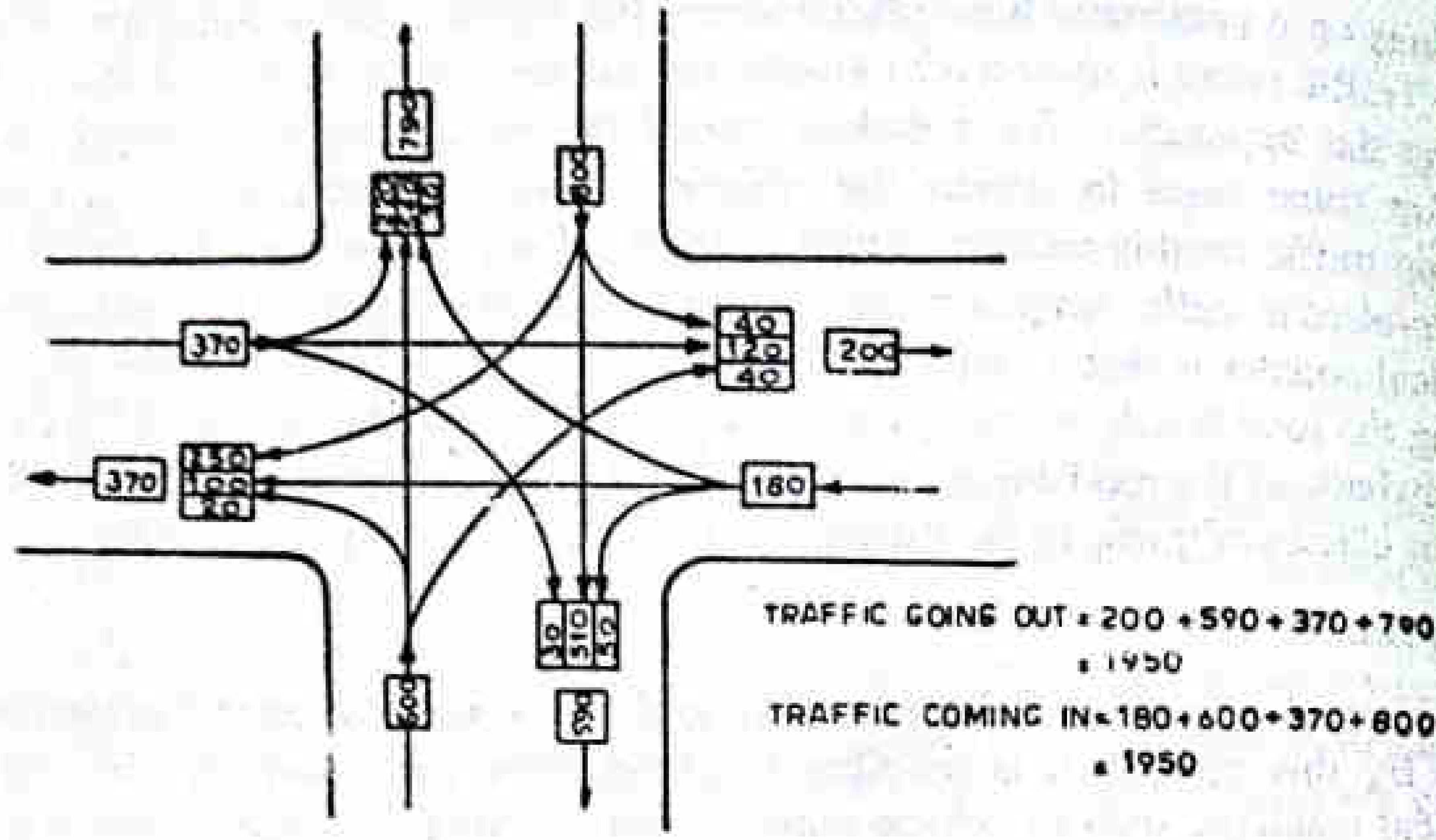


Fig. 5.2 Traffic Flow at Intersection

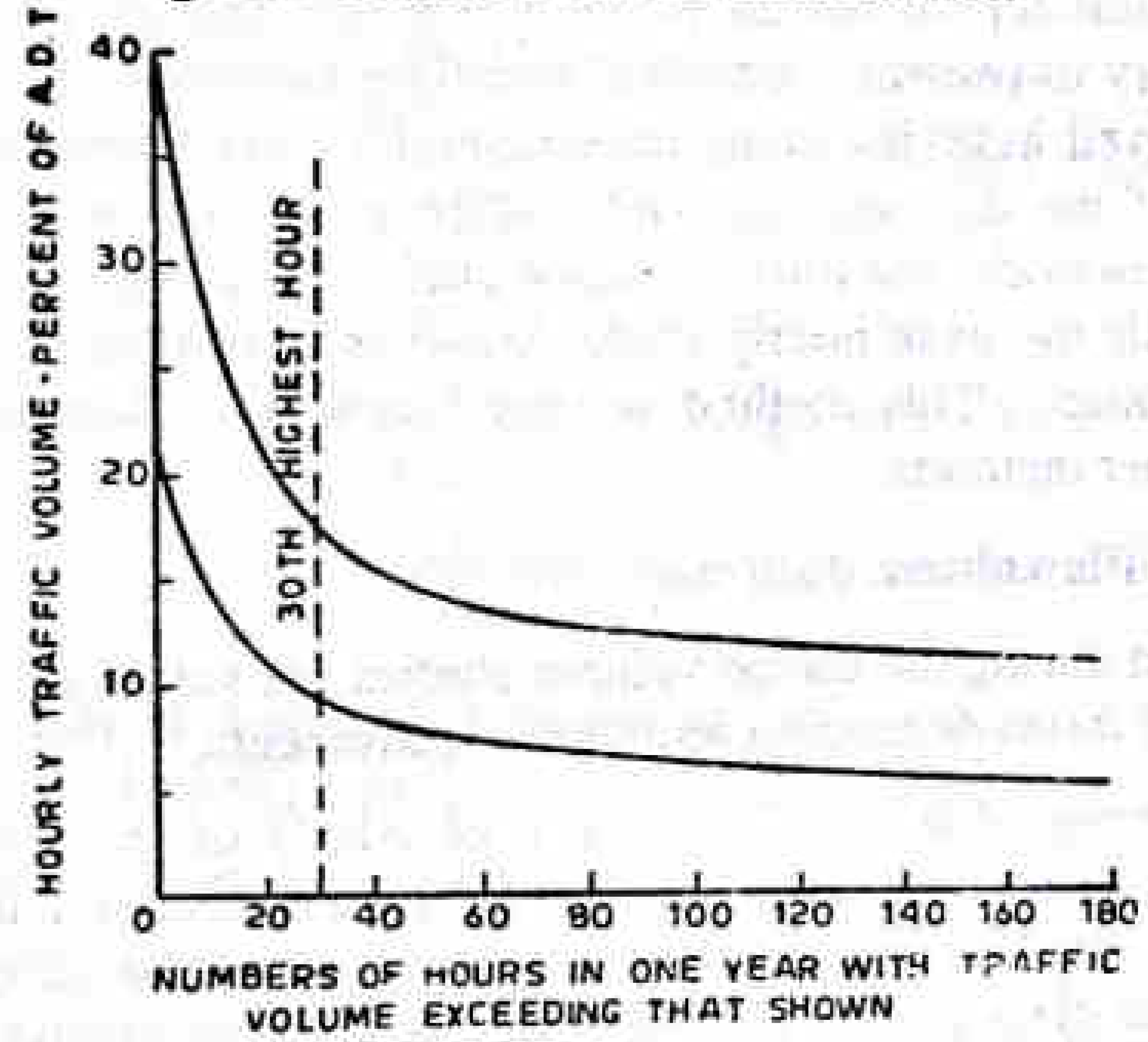


Fig. 5.3 Hourly Traffic Volumes

than this value. The highest or peak hourly volume of the year will be too high that it will not be economical to design the facilities according to this volume. The annual average hourly volume (AAHV) found from AADT will not be sufficient during considerable period of an year. The high facilities designed with capacity for 30th highest hourly traffic volume in the assumed year is found to be satisfactory from both facility and economic considerations. This is because the cost will be less when compared to the peak hourly volume and hence reasonable. There will be congestion only during 29 hours in the year. Thus the thirtieth highest hourly volume is generally taken as the hourly volume for design. However the actual design hourly volume may be decided by drawing the diagram as shown in Fig. 5.3, after carrying out traffic volume studies on the desired location of the road. The design hourly volume thus arrived at need not necessarily be the thirtieth highest hourly volume in all the cases.

Speed studies

The actual speed of vehicles over a particular route may fluctuate widely depending on several factors such as geometric features, traffic conditions, time, place, environment and driver.

Travel time is the reciprocal of speed and is a simple measure of how all well a road network is operating.

Spot speed is the instantaneous speed of a vehicle at a specified section or location.

Average speed is the average of the spot speeds of all vehicles passing a given point on the highway.

There are two definitions for the average of a series of spot speed measurements viz.; *space-mean speed* and *time-mean speed.* Space-mean speed represents the average speed of vehicles in a certain road length at any time. This is obtained from the observed t ravel time of the vehicles over a relatively long stretch of the road. Space-mean speed is calculated from :

$$V_s = \frac{3.6\,d\,n}{\sum_{i=1}^{n} t_i}$$

where V_s = space-mean speed, kmph

d = length of road, considered, m

n = number of individual vehicle observations

t_i = observed travel time (sec) for ith vehicle to travel distance d, m

The average travel time of all the vehicles is obtained from the reciprocal of space-mean speed.

Time-mean speed represents the speed distribution of vehicles at a point on the roadway and it is the average of instantaneous speeds of observed vehicles at the spot. Time-mean speed is calculated from :

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

where V_t = time-mean speed, kmph

V_i = observed instantaneous speed of ith vehicles, kmph

n = number of vehicles observed

The space-mean speed is slightly lower than time-mean speed under typical speed conditions on rural highways.

Running speed is the average speed maintained by a vehicle over a particular stretch of road, while the vehicle is in motion; this is obtained by dividing the distance covered by the time during which the vehicle is actually in motion.

Overall speed or travel speed is the effective speed with which a vehicle traverses a particular route between two terminals; this is obtained by dividing the total distance travelled by the total time taken including all delays and stoppages enroute.

Speed studies carried out occasionally give the general trend in speeds. There are two types of speed studies carried out,

(i) Spot speed study, and

(ii) Speed and delay study

Spot speed study

Spot speed study may be useful in any of the following aspects of traffic engineering :

(a) to use in planning traffic control and in traffic regulations.

(b) to use in geometric design-for redesigning existing highways or for deciding design speed for new facilities.

(c) to use in accident studies

(d) to study the traffic capacity

(e) to decide the speed trends

(f) to compare diverse types of drivers and vehicles under specified conditions.

The spot speeds are affected by physical features of the road like pavement width, curve, sight distance, gradient, pavement unevenness intersections and road side developments. Other factors affecting spot speeds are environmental conditions (like weather, visibility), enforcement, traffic conditions, driver, vehicle and motive of travel.

There are a number of methods to measure spot speed. The spot speed may be obtained either by finding the running speed of vehicles over a short distance of less than 50 metre or by finding the instantaneous speed while crossing a section, depending on the method used. The spot speeds of a few typical sample of vehicles are found during the sampling periods of the day, days of the week and months of the year.

One of the simplest methods of finding spot speed is by using *enoscope* which is just a mirror box supported on a tripod stand. In its simplest principle, the observer is stationed on one side of the road and starts a stopwatch when a vehicle crosses that section. An enoscope is placed at a convenient distance of say 30 m in such a way that the image of the vehicle is seen by the observer when the vehicle crosses the section where the enoscope is fixed (see Fig. 5.4) and at this instant the stop watch is stopped. Thus the time required for the vehicle to cross the known length is found and is converted to the speed in kmph. The main advantage of this method is that it is a simple and cheap equipment and is easy to use. The greatest disadvantage is that the progress is so slow as it is difficult to spot out typical vehicles and the number of samples observed will be less. There is also a possibility of human error.

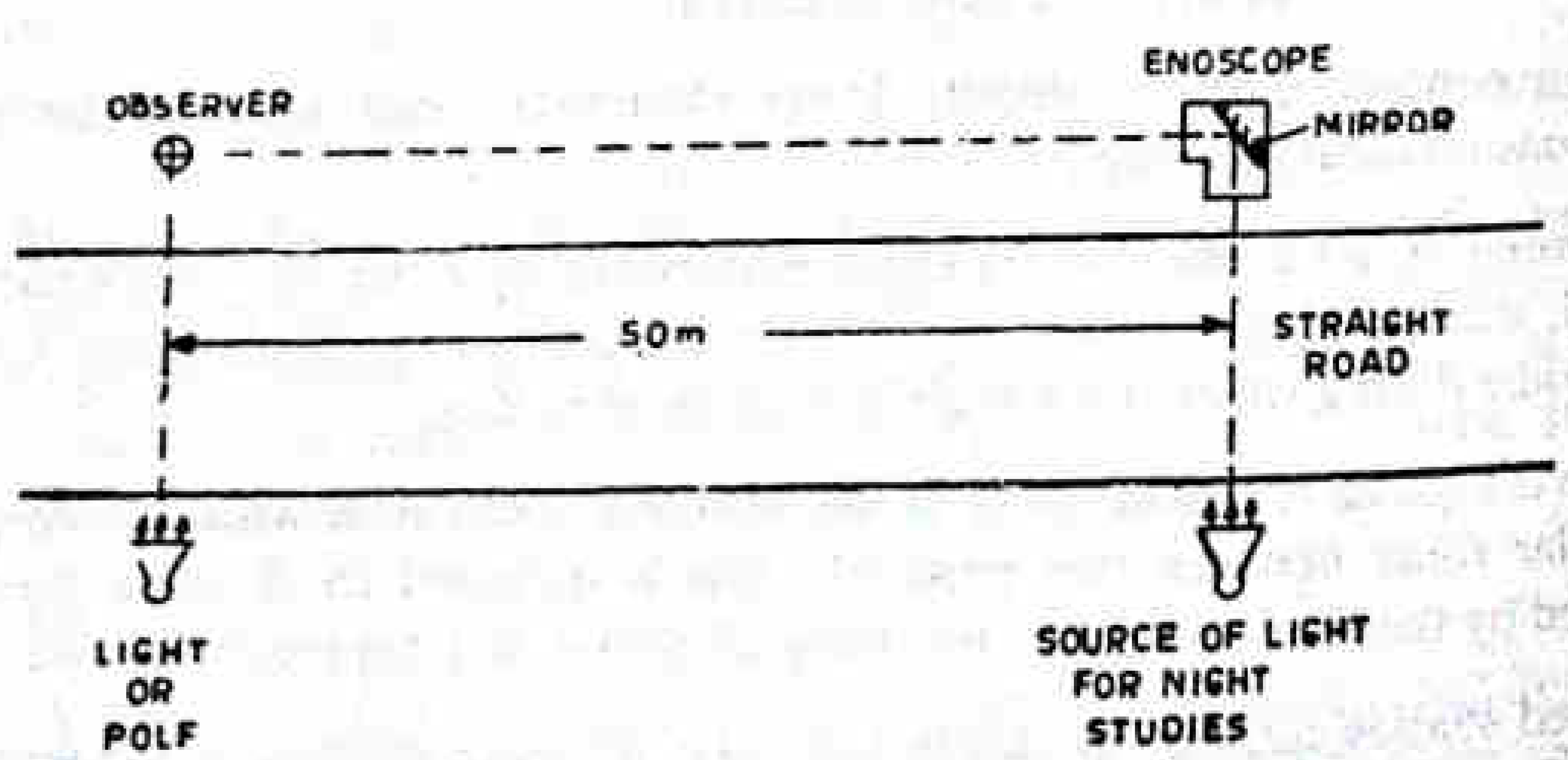


Fig. 5.4 Spot Speed by Enoscope

Other equipment used for spot speed measurements are graphic recorder, electronic meter, photo electric meter, radar, speed meter and by photographic methods. Of all these methods, the radar speed meter method seems to be the most efficient one as it is capable of measuring the spot speeds instantaneously and also record them automatically. But this equipment is costly.

Presentation of spot speed data

(a) *Average speed of vehicles* : From the spot speed data of the selected samples, frequency distribution tables are prepared by arranging the data in groups covering various speed ranges and the number of vehicles in such range. The *arithmetic mean* is taken as the average speed. The table gives the general information of the speeds maintained on the section, and also regarding the speed distribution pattern.

(b) *Cumulative speed of vehicles* : A graph is plotted with the average values of each speed group on the X-axis and the cumulative percent of vehicles travelled at or below the different speeds on the Y-axis. From this graph, the 85th *percentile speed* is found out which gives that speed at or below 85 percent of the vehicles are passing the point on the highway or only 15 per cent of the vehicles exceed the speed at that spot. See Fig. 5.5. The drivers exceeding 85th percentile speed are usually considered to drive faster than the safe speed under existing conditions and hence this speed is adopted for the *safe speed limit* at this zone. However for the purpose of highway geometric design, the 98th percentile speed is taken.

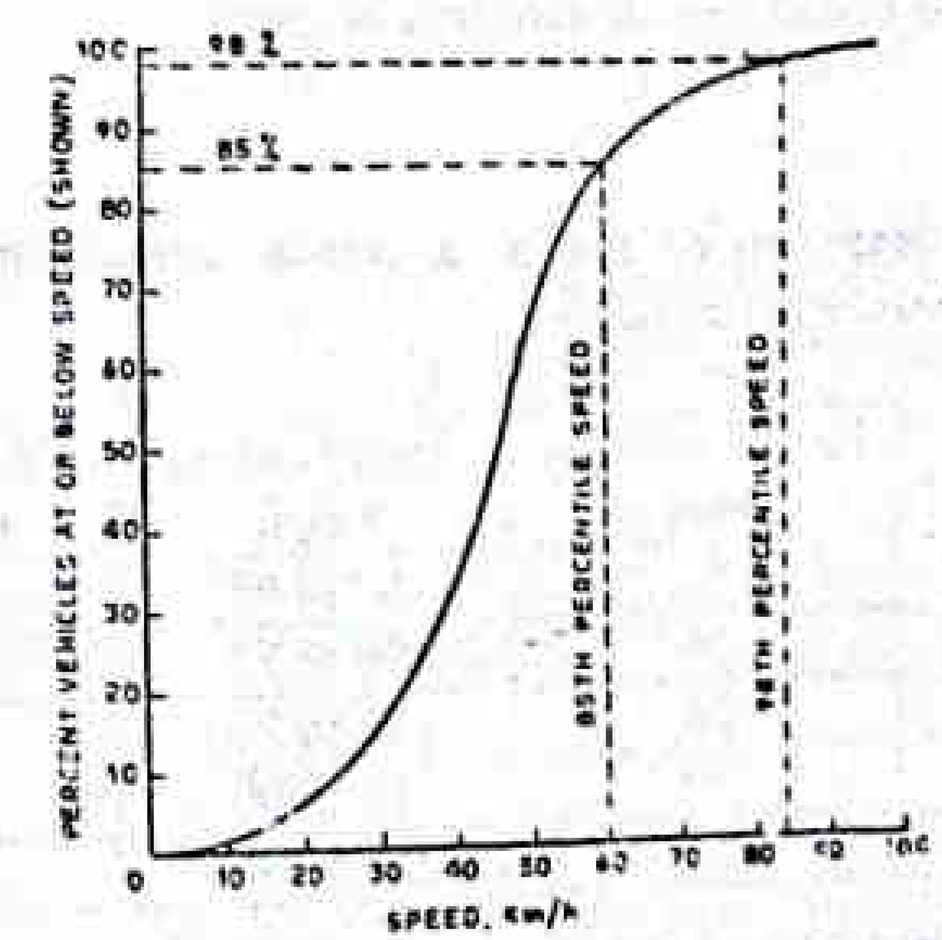


Fig. 5.5 Cumulative Speed Distribution

The 15th percentile speed represents the lower speed limit if it is desired to prohibit slow moving vehicles to decrease delay and congestion, as 85 percent of the vehicles to the stream travel at speeds higher than this value and therefore need overtaking opportunities.

(c) *Modal average* : A frequency distribution curve of spot speeds is plotted with speed of vehicles or average values of each speed group of vehicles on the X-axis and the percentage of vehicles in that group on the Y-axis. See Fig. 5.6. This graph is called the speed distribution curve. This curve will have a definite peak value of travel speed across the section and this speed is denoted as model speed. The speed distribution curve is helpful in determining the speed at which the greatest proportion of vehicles move, given by the model speed.

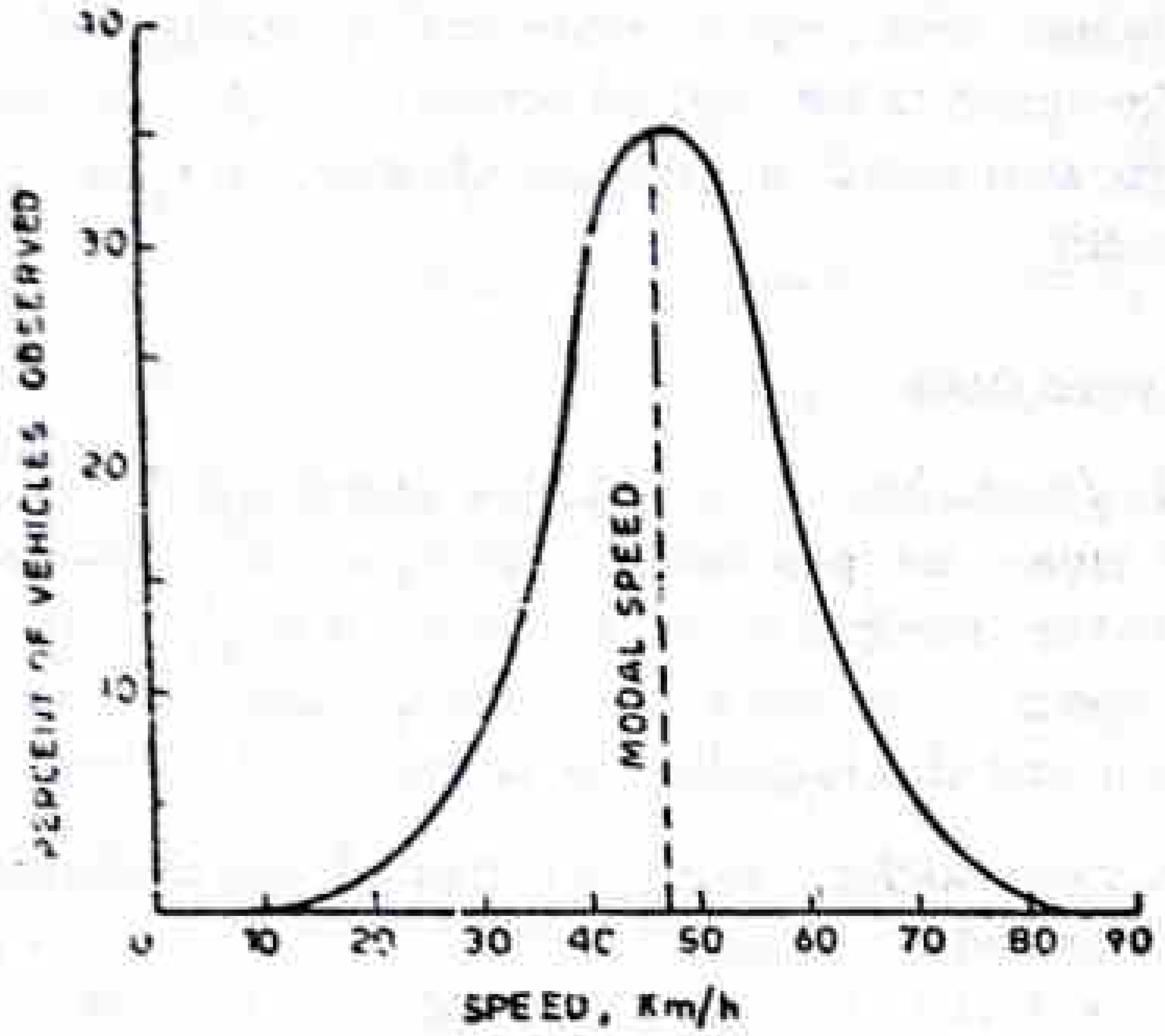


Fig. 5.6 Frequency Distribution Curve of the Spot Speeds

All vehicles do not travel at the same speed at a location along a road. The amount of speed dispersion or the spread from the average speed affects both capacity and safety. For free flow of vehicles, the speed distribution follows a normal distribution curve. The quality of flow of vehicles in a stream therefore depends on the speed dispersion. This may be judged by several methods such as (85^{th} minus 15^{th} percentile speeds); Standard deviation of speeds, or the coefficient of variation in speed.

Example 5.6

Spot speed studies were carried out at a certain stretch of a highway and the consolidated data collected are 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 (i) the upper and lower values or speed limits for regulation of mixed traffic flow and (ii) the design speed for checking the geometric design elements of the highway.

Solution

The problem may be solved in three stages. First a frequency distribution table is prepared, next the cumulative frequency distribution curve is drawn and finally the appropriate values are obtained from the curve. Refer table 5.2. Column no.2 represents the average values of the different speed ranges. The number of vehicles observed in each speed range is represented as the frequency f in column no.3. The percentage frequency values given in Column no. 4 are based on the total number of vehicles observed in all the speed ranges. The cumulative values of percentage frequency are given in column no.5.

Table 5.2 Frequency Distribution of Spot Speed Data (Example 5.6)

Speed range, Kmph	Mid speed, kmph	Frequency, f	Frequency, %	Cumulative Frequency, %
1	2	3	4	5
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	

Using the values of mid-speed and cumulative frequency % column 2 and 5 of Table 5.2, cumulative speed distribution curve is plotted (see Fig. 5.5); from this graph the following results are obtained :

(i) Upper speed limit for regulation = 85^{th} percentile speed = 60 kmph

(ii) Lower speed limit for regulation = 15^{th} percentile speed = 30 kmph

(iii) Speed to check design elements = 98^{th} percentile speed = 84 kmph

Example 5.7

The table below gives the consolidated data of spot speed studies on a section of a road. Determine the most preferred speed at which maximum proportion of vehicles travels.

Speed range, kmph	No. of speed observations	Speed range, kmph	No. of speed observations
0 – 10	0	50 – 60	216
10 – 20	11	60 – 70	68
20 – 30	30	70 – 80	24
30 – 40	105	80 – 90	0
40 – 50	233		

Solution

The most preferred speed at which maximum proportion of vehicles travel is the modal speed which can be obtained by plotting the frequency distribution curve. The frequency distribution table of spot speeds prepared for this purpose is given in Table 5.3.

The frequency distribution curve is plotted (see Fig. 5.6) using the mean speed and the percent frequency values of the Table 5.3. The modal speed corresponds to the maximum value of percentage frequency in Fig. 5.6 and is equal to 47 kmph.

Table 5.3 Frequency Distribution of Spot Speed Data (Example 5.7)

Speed range, kmph	Mean speed, kmph	Frequency, f	Percent frequency
0 – 10	5	0	0.0
10 – 20	15	11	1.6
20 – 30	25	30	4.4
30 – 40	35	105	15.3
40 – 50	45	233	33.9
50 – 60	55	216	31.4
60 – 70	65	68	9.9
70 – 80	75	24	3.5
80 – 90	85	0	0.0
	Total :	687	105.0

Speed and delay study

The speed and delay studies give the running speeds, overall speeds, fluctuations in speeds and the delay between two stations of a road spaced far apart. They also give the information such as the amount, location, duration frequency and causes of the delay in the traffic stream. The results of the speed and delay studies are useful in detecting the spots of congestion, the causes and in arriving at a suitable remedial measures. The studies are also utilised in finding the travel time and in *benefit-cost analysis*. In general the efficiency of the roadway be judged from the travel time.

The delay or the time lost by traffic during the travel period may be either due to fixed delays or operational delays. Fixed delay occurs primarily at intersections due to traffic signals and at level crossings. Operational delays are caused by the interference of traffic movements, such as turning vehicles, parking and imparking vehicles, pedestrians etc., and by internal friction in the traffic stream due to high traffic volume, insufficient capacity and by accidents. Therefore the overall travel speed between the origin and destination points of travel is invariably lower than the desired running speed.

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

Floating car or riding check method

License plate or vehicle number method

Interview technique

Elevated observations, and

Photographic technique

In the *floating car method* a test vehicle is driven over a given course of travel at approximately the average speed of the stream, thus trying to *float* with the traffic stream. A number of test runs are made along the study stretch and a group of observers record the various details. One observer is seated in the floating car with two stop watches. One of the stop watches is used to record the time at various control points like intersections, bridges or any other fixed points in each trip. The other stop watch is used to find the duration of individual delays. The time, location and cause of these delays are recorded by the second observer either on suitable tabular forms or by voice recording equipment. The number of vehicle overtaking the test vehicle and that overtaken by the test vehicles are noted in each trip by a third observer. The number of vehicles travelling in the opposite direction in each trip is noted by a fourth observer. However in mixed traffic flow, more number of observers will be required to count the vehicles of different classes. In this method the detailed information is obtained concerning all phases of speed and delay including location, duration and causes of delay.

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

$$\bar{t} = t_w - n_y/q \tag{5.1}$$

$$q = \frac{n_a + n_y}{t_a + t_w} \tag{5.2}$$

where

q = flow of vehicles (volume per min), in one direction of the stream

n_a = average number of vehicles counted in the direction of the stream when the test vehicle travels in the opposite direction

n_y = the average number of vehicles overtaking the test vehicle minus the number of vehicles overtaken when the test is in the direction of q

t_w = average journey time, in minute when the test vehicle is travelling with the stream q

t_a = average journey time, in minute when test vehicle is running against the stream q

In the *license plate or vehicle number method*, synchronized stop watches or voice recording equipment are used. Observers are stationed at the entrance and exit of a test section where information of travel time is required. The timings and the vehicle numbers are noted by the observers of the selected sample. From the office computations, travel time of each vehicles could be found. But the method does not give important details such as causes of delays and the duration and number of delays within the test section.

In the *interview technique*, the work can be completed in a short time by interviewing and collecting details from the road users on the spot. However the data collected may not provide with all the details correctly.

Elevated observation and *photographic technique* are useful for studying short test sections like intersection etc.

Intersection delays studies need special attention as this poses a major problem to the traffic engineer. Such studies at each intersection will help in evaluating the efficiency and effectiveness of the control device like signal system, the remedial measures for accidents etc.

Example 5.8

The consolidated data collected from speed and delay studies by floating car method on a stretch of urban road of length 3.5 km, running North-South are given below. Determine the average values of volume, journey speed and running speed of the traffic stream along either direction.

Trip No.	Direction of trip	Journey time Min. Sec.	Total stopped delay Min. Sec.	No. of vehicles overtaking	No. of vehicles overtaken	No. of vehicles from opp. direction
1	N – S	6 – 32	1 – 40	4	7	268
2	S – N	7 – 14	1 – 50	5	3	186
3	N – S	6 – 50	1 – 30	5	3	280
4	S – N	7 – 40	2 – 00	2	1	200
5	N – S	6 – 10	1 – 10	3	5	250
6	S – N	8 – 00	2 – 22	2	2	170
7	N – S	6 – 28	1 – 40	2	5	290
8	S – N	7 – 30	1 – 40	3	2	160

Solution

The mean values of journey time, stopped delay, number of vehicles overtaking, overtaken and in opposite direction for North-South and South-North directions are obtained from Table 5.4.

Table 5.4 Mean values of speed and Delay Data (Example 5.8)

Direction	Journey time Min. Sec.	Stopped delay Min. Sec.	Number of vehicles		
			Overtaking	Overtaken	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 – 28	1 – 40	2	5	290
Total :	26 – 00	6 – 00	14	20	1088
Mean :	6 – 30	1 – 30	3.5	5.0	272
S – N	7 – 14	1 – 50	5	3	186
	7 – 40	2 – 00	2	1	200
	8 – 00	2 – 22	2	2	170
	7 – 30	1 – 40	3	2	160
Total :	30 – 24	7 – 12	12	8	716
Mean :	7 – 36	! – 40	3.0	2.0	179

(i) North-South direction

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

$$q = \text{average volume} = \frac{a_a + n_y}{t_a + t_w} = \frac{179 - 1.5}{7.6 + 6.5} = 12.59 \text{ veh/min}$$

$$\bar{t} = \text{average journey time} = t_w - \frac{n_y}{q} = 6.5 - \frac{(-1.5)}{12.59} = 6.62 \text{ min}$$

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

Average stopped delay = 1.5 min

Average running time = Average journey time – average stopped delay

= 6.621 – 1.50 = 5.12 min

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

(ii) South-North direction

n_y = 3.0 – 2.0 = 1.0

t_w = 7.6 min

t_a = 6.5 min

n_a = (from N – S strips) = 272

$$q = \frac{272 + 1.0}{6.5 + 7.6} = 19.36 \text{ veh/min}$$

$$\bar{t} = 7.6 - \frac{1.0}{19.36} = 7.55 \text{ min}$$

Journey speed = $\frac{3.5 \times 60}{7.55}$ = 27.8 kmph

Average stopped delay = 1.8 min

Average running time = 7.55 – 1.80 = 5.75 min

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

Origin and Destination Studies

The origin and destination (O & D) study is carried out mainly to (i) plan the road network and other facilities for vehicular traffic and (ii) plan the schedule of different modes of transportation for the trip demand of commuters.

The O & D studies of vehicular traffic determines their number, their origin and destination in each zone under study. The data may also be supplemented by the number of passengers in each vehicle, purpose of each trip, intermediate stops made and reasons etc. Origin and destination study gives informations like the actual direction of travel, selection of routes and length of the trip. These studies are most essential in planning new highway facilities and in improving some of the existing systems. As an example there can be a high percentage of through traffic which may be diverted by providing a by-pass and thus considerable saving in distance and time can be made. O & D study provides the basic data for determining the desired directions of flow or the *desire lines*. This is considered to be one of the important traffic studies needed to solve many traffic problems in a zone and the most important study to plan the highway system in a region.

Scientific planning of transportation system and mass transit facilities in cities should be based on O & D data of passenger trips. Also future traffic needs may by estimated by extrapolating the data from O & D study, together with socio-economic studies. (See Art. 5.2.5 for estimation of future traffic).

The various applications of O & D studies may be summed up as follows :

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

(ii) to plan transportation system and mass transit facilities in cities including routes and schedules of operation.

(iii) to locate *expressway* or major routes along the desire lines.

(iv) to establish preferential routes for various categories of vehicle including by-pass.

(v) to locate terminals and to plan terminal facilities.

(vi) to locate new bridge as per traffic demands.

(vii) to locate intermediate stops of public transport.

(viii) to establish design standards for the road, bridges and culverts along the route.

There are a number of methods for collecting the O & D data. Some of the methods, commonly adopted are :

Road-side interview method, License plate method

Return post card method, Tag-on-car method and

Home interview method.

The choice of the method is made judiciously depending on the objective and location.

Road side interview method

The vehicles are stopped at previously decided interview stations, by a group of persons and the answers to prescribed questionnaire are collected on the spot. The information collected include the place and time of origin and destination, route, locations of stoppages, the purpose of the trip, type of vehicle and numbers of passengers in each vehicle. The traffic may be filtered through a prescribed lane by previous warning signs and with the help of police so that each driver of the selected sample of vehicles is interviewed. The percentage of sample interviewed out of the total traffic in each selected period should also be noted from appropriate traffic volume study taken simultaneously.

In this method the data is collected quickly in short duration and the field organisation is simple and the team can be trained quickly. The main drawback of the method is that the vehicles are stopped for interview, and there is delay to the vehicular movement. Also resentment is likely from the road users. Further, unless there is enough space, undue congestion may result due to stopped vehicles.

License plate method

The entire area under study is cordoned out and the observers are simultaneously stationed at all points of entry and exit on all the routes leading to and out of the area. Each party at the observation station is given synchronized time pieces and they note the license plate numbers (registration numbers) of the vehicles entering and leaving the cordoned area and the time. Separate recording sheets are maintained for each direction of movement for a specified time interval. After collecting the field data major work remains of the office computations and analysis, by tracking each vehicle number and its time of entering and leaving the cordoned area.

This method is quite easy and quick as far as the field work is concerned. The field organisation can also be trained quickly. The method however involves a lot of office computations in tracing the trips through a net work of stations. Unless there is a large net work of stations to take observations along the route of the vehicle, it is not easy to get the information of the routes followed by the vehicles.

Hence a large number of teams are required to take simultaneous observations when a large area is to be surveyed. However, this method is quite advantageous when the area under consideration is small, like a large intersection or a small business centre.

Return post card method

Pre-paid business reply post cards with return address are distributed to the road users at some selected points along the route or the cards are mailed to the owners of vehicles. The questionnaire to be filled in by the road user is printed on the card, along with a request for co-operation and purpose of the study. The distributing stations for the cards may be selected where vehicles have to stop as in case of a toll booth.

The method is suitable where the traffic is heavy. The personnel need not be skilled or trained just for distributing the cards. Only a part of the road users may return the cards promptly after filling in the desired details properly and correctly. If conclusions are drawn in such cases, it is likely that these may not give a true picture.

Tag on car method

In this method a pre-coded card is stuck on the vehicle as it enters the area under study. When the car leaves the cordon area the other observations are recorded on the tag. This method is useful where the traffic is heavy and moves continuously. But the method gives only information regarding the points of entry and exit and the time taken to traverse the area.

Home interview method

A random sample of 0.5 to 10 percent of the population it selected and the residences are visited by the trained personal who collect the travel data from each member of the house hold. Detailed information regarding the trips made by the members is obtained on the spot. The data collected may be useful either for planning the road net work and other roadway facilities for the vehicular traffic or for planning the mass transportation requirements of the passengers. The problem of stopping vehicle and consequent difficulties are avoided altogether. The present travel needs are clearly known and the analysis is also simple. Additional data including socio-economic and other details may be collected so as to be useful for forecasting traffic and transportation growth. But to have complete coverage of the entire cross section of the population is very tedious.

While planning for O and D studies at a place, it is necessary to decide the method of study. The selection of the method is dependent on the objective and the location. The influence of year and dates of study on the type and amount of traffic demand should be known. Care is needed in selecting the method of sampling and mode of data collection. The sample size should be decided keeping in view the desired accuracy and cost.

Work spot interview method

The transportation needs of work trips can be planned by collecting the O & D data at work spots like the offices, factories, educational institutions, etc. by personal interviews.

Presentation of O and D data

The data are presented in the following forms :

(i) Origin and destination tables are prepared showing number of trips between different zones.

(ii) *Desire lines* are plotted which is a graphical representation prepared in almost all O and D surveys. Desire lines are straight lines connecting the origin points with destinations, summarized into different area groups (See Fig. 5.7). The width of such desire lines is drawn proportional to the number of trips in both directions. The desire line density map easily enable to decide the actual desire of the road users and thus helps to find the necessity of a new road link, a diversion, a by-pass or a new bridge. These desire lines may be compared with the existing flow pattern

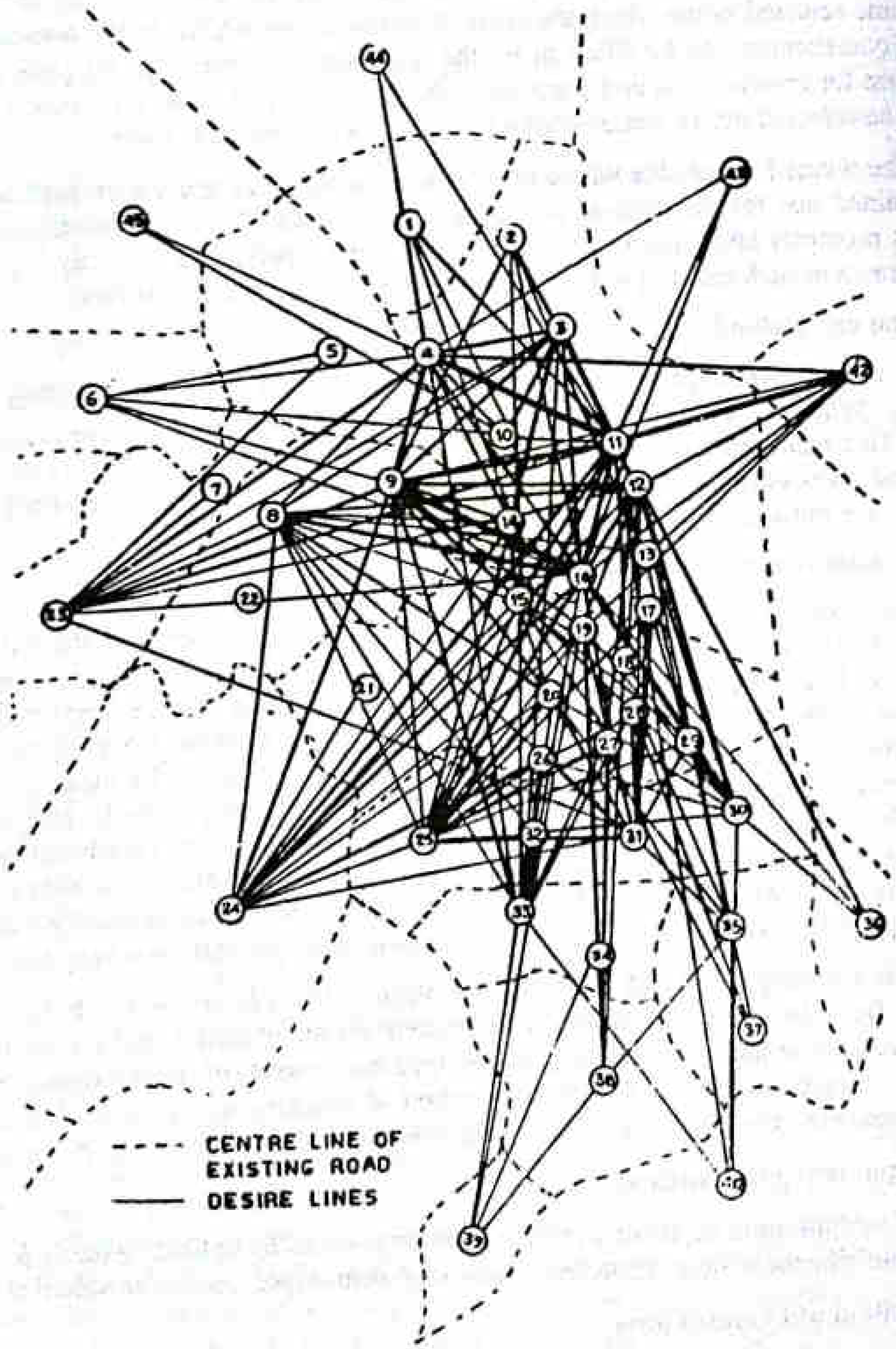


Fig. 5.7 Desire Lines

along the existing routes by superimposing one over the other with the help of tracing sheets.

(iii) The relative magnitude of the generated traffic and geometrical relationships of the zones involved may be represented by *pie charts*, in which circles are drawn, the diameter being proportional to the number of trips.

(iv) Contour lines may be plotted similar to topographic contours. The shape of the contours would indicate the general traffic need of the area.

Traffic flow characteristics and studies

Traffic stream generally has flow and counter flow along a common route, unless the stream is separated into pair of one-way flows by proper design or regulation.

The basic traffic manoeuvres are *diverging, merging and crossing* as shown in Fig. 5.8. Of all these, diverging on the left is the easiest movement causing least problem of the traffic conflicts. This is because the traffic is regulated on the left side. Merging from the left side also does not cause much of conflict. But diverging to the right and also merging from the right create conflicts and hazard to the traffic moving in the straight path. Transfer of a vehicle from one traffic lane to the next adjacent traffic lane is called *lane change* and this involves diverging and merging.

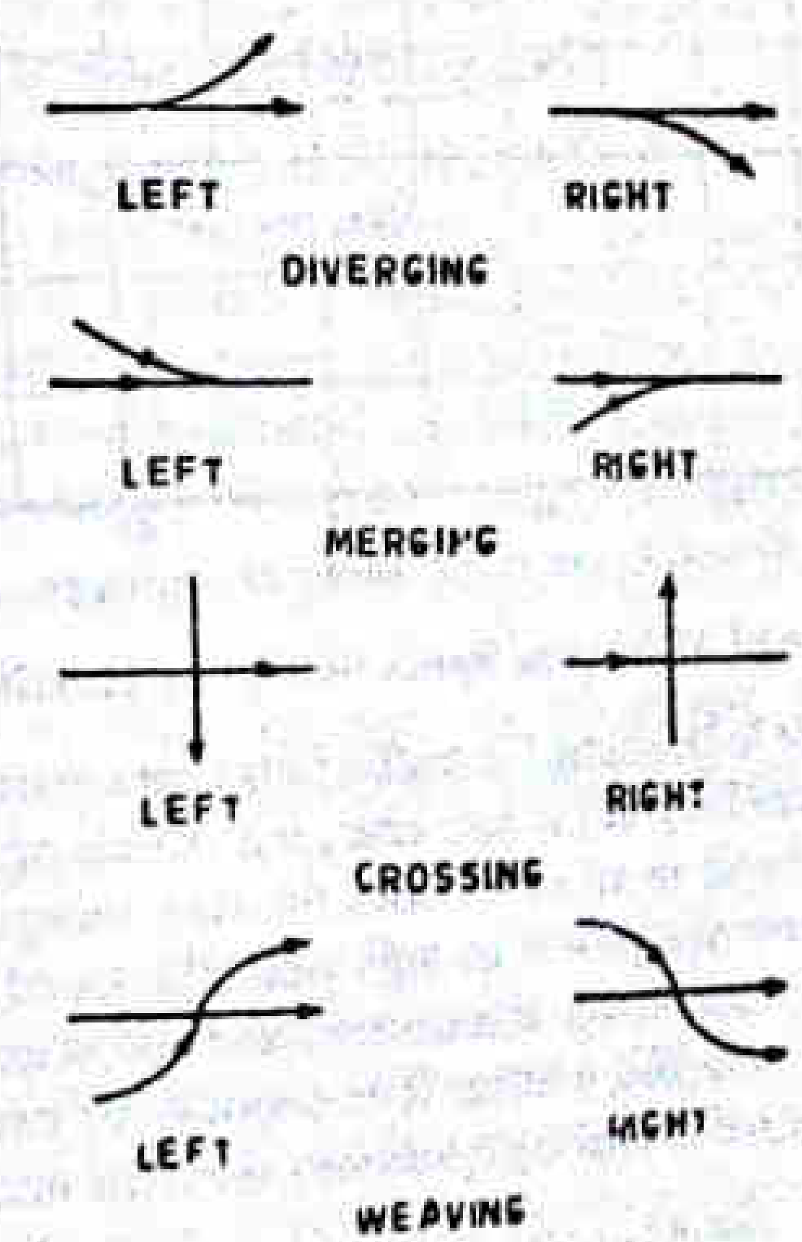


Fig. 5.8 Traffic Manoeuvres

The crossing traffic is the greatest problem in case of road intersections at level, because vehicles on one road have to stop before allowing the crossing stream of vehicles to cross their path. Thus the traffic capacity of two cross roads practically reduces to that of any one of the crossing roads or even lesser at the intersection. When a vehicle moves obliquely across the path of another vehicle moving in the same direction, at relatively small angle of crossing, the action is termed as *weaving*. The weaving manoeuvre may also be considered to consist of merging and diverging operations, along the stretch.

In two-way movements there may be crossing and over taking manoeuvres. The traffic stream characteristics are affected by the wide range of vehicles and road users, geometric feature of the road and intersections and other regulatory measures. Further the environmental conditions also affect the traffic stream flow.

The points to be particularly studied in traffic flow are the transverse and longitudinal distribution of vehicles on the various routes. The gaps ahead of each vehicle determine the longitudinal distribution of vehicles in one-way stream. See Fig. 5.9. 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*. The distance between successive vehicles moving in the same line measured from head at any instance is the *space headway* or the spacing of the vehicles in the stream. The variations in headway show the longitudinal distribution of the stream. 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 the traffic stream, the minimum space headway increases where as the minimum time headway first decreases and after reaching a minimum value at optimum speed on the stream, increases as shown in Fig. 5.9. Maximum flow or capacity flow is attained at this speed when the time headway is minimum.

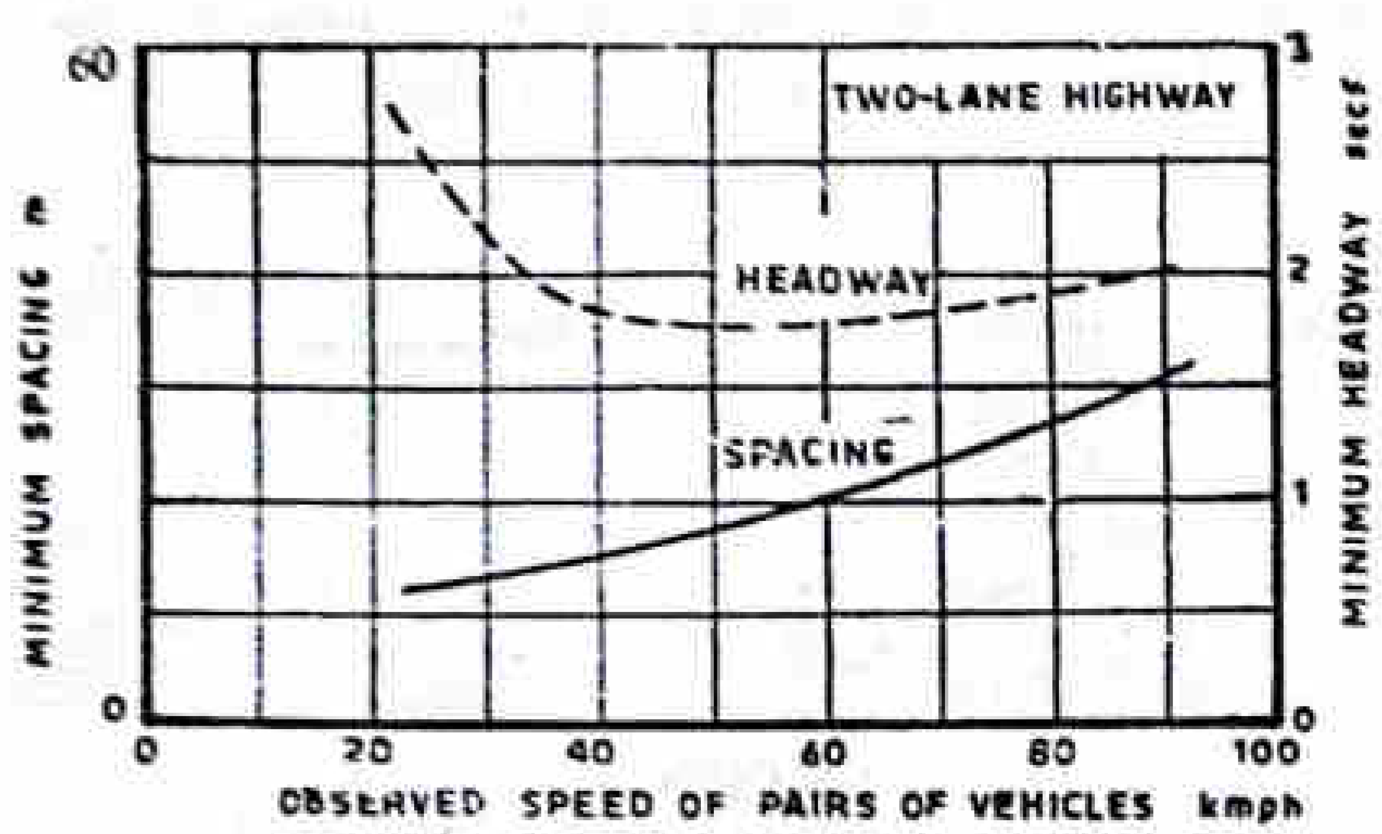


Fig. 5.9 Variation of Minimum Space and Time Headways with Speed

Another important factor to be studied in traffic flow characteristics is lane change in the traffic stream. When the headway of the lane changing vehicle rapidly decreases to almost zero, the lane change is forced; in all other cases the lane change may be optional. The frequency of demand for lane change will be high when the speed range of vehicles in the traffic stream is high. The lane change manoeuvres and characteristics would very much depend on the number of lanes and whether it is one-way or two-way movement. The merging, diverging, weaving and overtaking operations, all come under lane changes.

Study of traffic flow characteristics includes both transverse and longitudinal distribution of vehicles in the traffic stream and this is useful in geometric design features such as traffic capacity, volume, number of lanes and width of carriageway. The study is also very much needed to decide traffic regulatory measure like one-way movements and for the design of traffic control methods. Traffic flow study is particularly essential for large intersections.

Traffic capacity studies

Before studying details of traffic capacity, it may be worth while to define some of the related terms which are often used.

Traffic volume is the number of vehicles moving in a specified direction on a given lane or roadway that pass a given point or cross section during specified unit of time. Traffic volume is expressed as vehicles per hour or vehicles per day.

Traffic density is the number of vehicles occupying a unit length of lane of roadway at a given instant, usually expressed as vehicles per kilometre. Traffic volume is the product of the traffic density and traffic speed.

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.

Traffic capacity is the ability of a roadway to accommodate traffic volume. It is expressed as the maximum number of vehicle in a lane or a road that can pass a given point in unit time, usually an hour, i.e., vehicles per hour per lane or roadway. Capacity and volume are measures of traffic flow and have the same units. 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 roadway. The capacity of a roadway depends on a number of prevailing roadway and traffic conditions.

Basic capacity is the maximum number of passenger cars 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 attained. Two roads having the same physical features will have the same basic capacity irrespective of traffic conditions, as they are assumed to be ideal. Thus basic capacity is the theoretical capacity.

Possible capacity 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 possible capacity of a road is generally much lower than the basic capacity as the prevailing roadway and traffic conditions are seldom ideal. In a worst case when the prevailing traffic condition is so bad that due to traffic congestion, the traffic may come to a stand still, the possible capacity of the road may approach zero.

When the prevailing roadway and traffic conditions approach the ideal conditions, the possible capacity would also approach the basic capacity. Thus the value of possible capacity varies from zero to basic capacity. For the purpose of design, neither basic capacity nor possible capacity can be adopted as they represent two extreme cases of roadway and traffic conditions.

Practical capacity 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 driver's freedom to manoeuvre under the prevailing roadway and traffic conditions. It is the practical capacity which is of primary interest to the designers who strive to provide adequate highway facilities and hence this is also called design capacity.

Determination of theoretical maximum capacity

An estimate of theoretical maximum or basic capacity of a single lane may be made from the relation :

$$C = \frac{1000\,V}{S} \tag{5.3}$$

Here, C = capacity of a single lane, vehicle per hour

V = speed, kmph

S = average centre to centre spacing of vehicles, when they follow one behind the other as a queue or space headway, m

Thus the capacity depends upon the speed V and spacing S. The average spacing S between centre to centre of vehicles is equal to the average length of vehicle plus the clear spacing between the vehicles in the stream. The minimum clear spacing between vehicles are allowed for safe stopping of the rear vehicle in case the vehicle ahead suddenly stops. It is always found that drivers follow the vehicle ahead at a closer gap at a lower speeds and the clear spacing is increased instinctively at higher speeds of the traffic stream (Fig. 5.9).

Thus the 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) traffic volume to capacity ratio of the road section at the instant or the level of service

(v) The proportion of vehicle classes in the stream

(vi) road geometrics and

(vii) environmental factors

The assumption that space gap increases in direct proportion with the speed of the vehicle or that of the traffic stream is therefore a very much simplified one and gives only an approximate average value of the space gap between vehicles in the traffic stream. The space gap allowed by the following vehicle in a traffic stream is some time assumed to be equal to the distance travelled during the reaction time of the driver, assuming that the braking distances of the lead and the following vehicles are approximately equal. If the reaction time is t sec., the minimum space gap S_g is given by :

$$S_g = v\,t = 0.278\ V\,t,\ m$$

where v and V are average speeds in m/sec and kmph

The minimum space headway S in a traffic stream is therefore equal to the minimum space gap plus average length of vehicle L in the stream

$$S = S_g + L = 0.278\ V\,t + L$$

In a stream flow, as the driver of the following vehicle is quite alert, the average reaction time is found to be low; this value is often assumed to be 0.70 to 0.75 sec. In this analysis of overtaking sight distance (Art. 4.3.3) the value of reaction time has been assumed as 0.7 sec. in the empirical relation for spacing, i.e.,

$$S = (0.7\ v + L) = (0.2\ V + L),\ m$$

Thus a suitable value of S may be adopted in Eq. 5.3 to estimate the theoretical capacity of a traffic lane with homogeneous traffic flow.

It has been observed (as explained earlier) that with increase in speed of the traffic stream, the time headway decreases and after reaching a minimum value at an optimum speed, starts increasing (See Fig. 5.9). The maximum theoretical capacity of a traffic lane may therefore be obtained if the minimum time headway H_t is known.

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

where C is the capacity, vehicles per hour (3600 second), and H_t is the minimum time headway in second.

The relationship between speed and maximum capacity of a traffic lane is shown in Fig. 5.10. The peak value of the theoretical maximum capacity if reached at an optimum speed. As the speed is increased further, the maximum capacity of the lane starts decreasing due to increase in headway at the speed range.

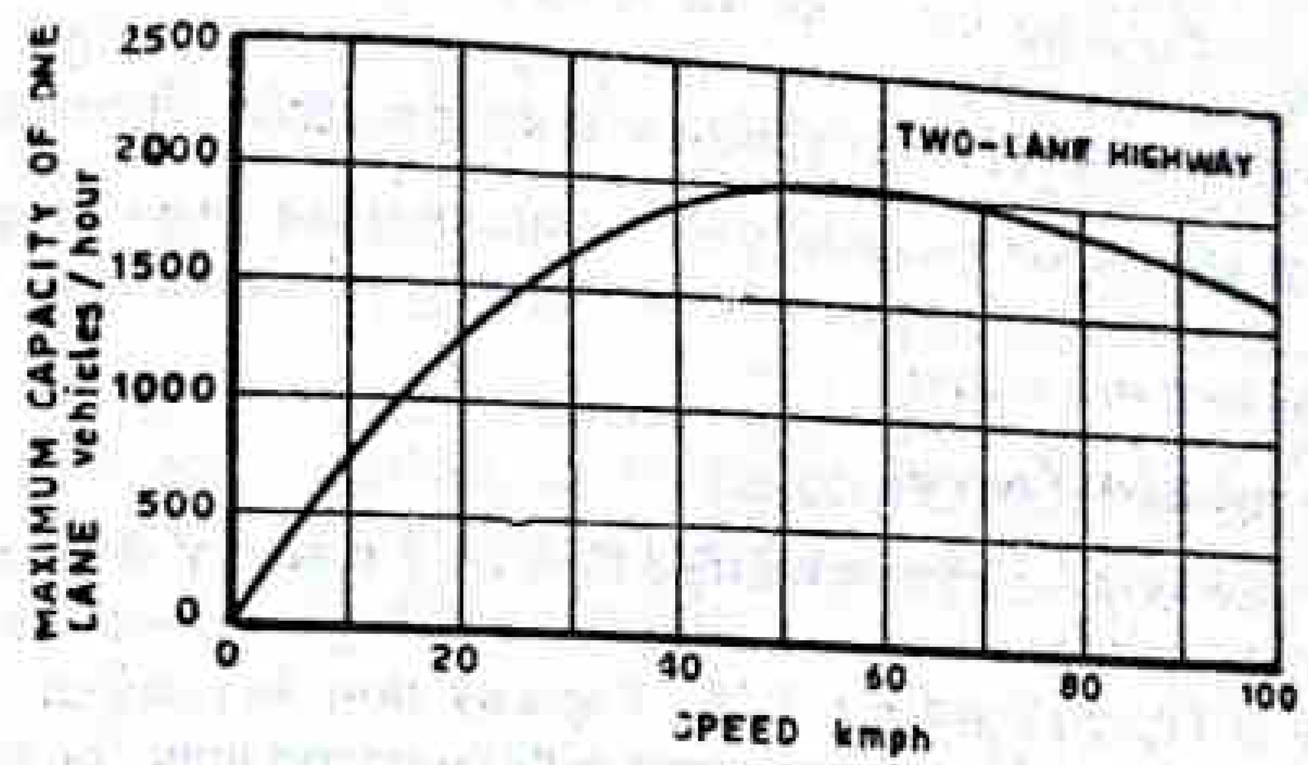


Fig. 5.10 Speed and Capacity

Factor affecting practical capacity

Some of the important factors that affect the practical capacity of a traffic lane are listed below :

(i) *Lane width* : As the lane width decreases, the capacity also decreases. The practical capacity of 3.0 m wide lane in a two-lane rural road may decrease to 76 percent of the capacity of a 3.5 m lane.

(ii) *Lateral clearance* : Vertical obstructions such as retaining walls, or parked vehicles near the traffic lane reduce the effective width of a lane and thus result in reduction in the capacity of lane. Further, restricted lateral clearance effects driving comfort and increases rates. A minimum clearance of 1.85 m from the pavement edge to the obstruction is considered desirable so that capacity is not affected adversely. When the distance from pavement edge to an obstruction decreases to 0.75 m on one side only, the capacity decreases to 96% and when this obstruction is on both sides, the percentage further decreases to 80% of the standard design capacity.

(iii) *Width of shoulders* : Narrow shoulders reduce the effective width of traffic lanes as the vehicles travel towards the centre of the pavement. When vehicles in emergency (like that of a tyre puncture or other break down) has to park on the shoulder of insufficient width, there is reduction in effective lane width resulting in a great reduction in the capacity of the lane.

(iv) *Commercial vehicles* : Large commercial vehicles like truck and buses occupy greater space and influence the other traffic in the same lane as well as the vehicles along the adjoining lanes. Also these heavy commercial vehicles may travel at lower speeds especially on grades.

(v) *Alignment* : If the alignment and geometrics are not upto the desired standards, the capacity will decrease. Particularly, restrictions to sight distance requirements cause reduction in capacity. Steep and long grades affect the capacity. When 60% of the road length has sub-standard OSD, the capacity decreases to 65% of the standard design capacity.

(vi) *Presence of intersections at grade* : Intersections restrict free flow of traffic and thus adversely affect the capacity. The capacity of an intersection of two roads crossing at grade will be slightly less than the road with lower capacity of the two. At signalized intersections as the vehicles have to stop alternately to allow crossing traffic, the capacity of the intersection will be further decreased. In order to provide consistent traffic flow and maximum capacity on important highways, is necessary to plan them as controlled access highways with grade separated intersections.

(vii) Other factors which affect the capacity are the stream speed, one or two way traffic movement, number of traffic lanes, vehicular and driver characteristics, composition of traffic and the traffic volume.

Design capacity and level of service

The capacity flow or the maximum possible flow on a roadway or a traffic lane is attained at particular optimum speed, the flow decreases at higher as well as lower speed values as shown in Fig. 5.11 and Fig. 5.18. Capacity flow is reached when all the vehicles flow as a stream at this optimum speed with no opportunity for overtaking; at this speed the level of service is considered to be fairly low when the volume of the road reaches the capacity or the volume to capacity ratio approaches a maximum possible value of 1.0.

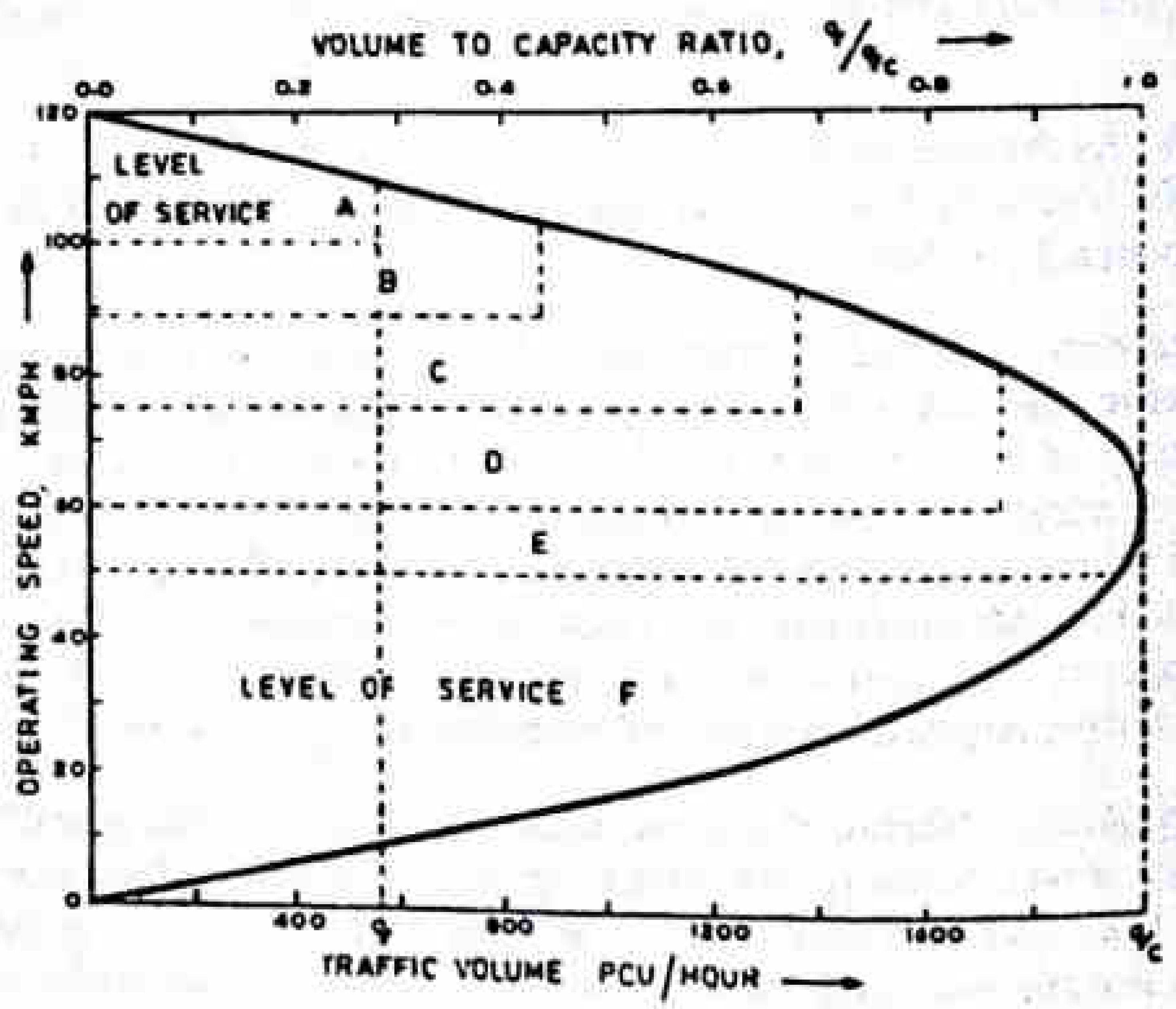


Fig. 5.11 General concept of level of service

Factors to be considered for the evaluation of level of service of a roadway in a comprehensive manner include the operating speed, travel time, traffic interruptions, freedom of manoeuvre, driving comfort, safety, economy etc. However, in order to simplify the level of service concept, two factors considered by the Highway Capacity Manual (HCM) are :

(i) the ratio of service volume to capacity q/q_c and

(ii) the operating or travel speed.

The HCM (Ref. 14) has suggested six levels of services A, B, C, D, E and F as shown in Fig. 5.11. Level of services A is considered to exist when the volume to capacity is so low that most of the individual vehicles have opportunities to travel at their oven desired speeds or free speeds and to overtake the slower vehicles at their will, this is possible at the highest speed range. With increase in the volume or the volume to capacity ratio, the operating speeds of faster vehicles and their opportunities to overtake decrease and the levels of service fall to decreasing values of B, C, D and E. Further, increase in vehicle arrivals causes further decrease in stream speed as well as in maximum flow, resulting in undue congestion and the lowest level of service F when forced flow conditions exist. The stream speed and the flow decrease to values much lower than the capacity flow condition and there will be practically no flow due to stopping of vehicles when the density attains the highest value or the 'jam density' as at a 'traffic jam'.

While designing the roadway facilities, care should be taken to adopt an appropriate value of design capacity, keeping in view the desired level of service. While planning and designing higher categories of highways, it is necessary to adopt design capacity values corresponding to high levels of service.

Passenger car unit (PCU)

Different classes of vehicles such as cars, vans, buses, trucks, auto rickshaw, motor cycles, pedal cycles, bullock carts, etc. are found to use the common roadway facilities without segregation on most of the roads in developing countries like India. The flow of traffic with unrestricted mixing of different vehicle classes on the roadways forms the heterogeneous traffic flow or the *mixed traffic flow*. The different vehicle classes have a wide range of static characteristics such as length, width etc. and dynamic characteristics such as speed, acceleration, etc. Apart from these, the driver behaviour of the different vehicle classes is also found to vary considerably. Therefore the mixed traffic flow characteristics are very much complex when compared to homogeneous traffic consisting of passenger cars only. It is rather difficult to estimate the traffic volume and capacity of roadway facilities under mixed traffic flow, unless the different vehicle classes are converted to one common standard vehicle unit. It is a common practice to consider the passenger car as the standard vehicle unit to convert the other vehicle classes and this unit is called Passenger Car Unit or PCU. Thus in mixed traffic flow, the traffic volume and capacity are generally expressed as PCU per hour or PCU/lane/hour and the traffic density as PCU per kilometre length of lane.

The PCU may be considered as a measure of the relative space requirement of a vehicle class compared to that of a passenger car under a specified set of roadway, traffic and other conditions. If the addition of one vehicle of a particular class in the traffic stream produces the same effect as that due to the addition of one passenger car, then that vehicle class is considered equivalent to the passenger car with a PCU value equal to 1.0. The PCU value of a vehicle class may be considered as the ratio of the capacity of a roadway when there are passenger cars only to the capacity of the same roadway when there are vehicles of that class only.

Factors affecting PCU values

The PCU values of different vehicle classes depend upon several factors. Some of these are listed below :

(i) Vehicles characteristics such as dimensions, power, speed, acceleration and braking characteristics.

(ii) Transverse and longitudinal gaps or clearances between moving vehicles which depends upon the speeds, driver characteristics and the vehicle classes at the adjoining spaces.

(iii) Traffic stream characteristics such as composition of different vehicle classes, mean speed and speed distribution of the mixed traffic stream, volume to capacity ratio etc.

(iv) Roadway characteristics such as road geometrics including gradient, curve, etc. access controls, rural or urban road, presence of intersections and the types of intersections.

(v) Regulation and control of traffic such as speed limit, one way traffic, presence of different traffic control devices, etc.

(vi) Environmental and climatic conditions.

Therefore the PCU value of a particular vehicle class may not remain a constant value as generally assumed. The important factors taken into account for a simple analysis of PCU values of different vehicle classes are :

(a) average speed of the vehicle class under the prevailing roadway and traffic conditions within the desired speed range.

(b) average length and width of the vehicle class.

(c) average transverse gap and longitudinal gap allowed between the vehicles of the same class in the speed range under consideration, during compact stream flow.

Based on the above factors, three different sets of PCU values have been worked out for :

(i) urban roads, mid block sections

(ii) signalized intersections and

(iii) kerb parking.

These are presented in Table 5.5 (See Ref. 24).

Table 5.5 Suggested PCU value for urban roads

S. No.	Vehicles class	PCU values of vehicle classes at :		
		(i) Urban roads, mid-block sections	(ii) Signalised intersection	(iii) Kerb parking (parallel & angle)
1.	Car	1.0	1.0	1.0
2.	Bus and truck	2.2	2.8	3.4
3.	Auto rickshaw	0.5	0.4	0.4
4.	Two wheeler automobile	0.4	0.3	0.2
5.	Pedal cycle	0.7	0.4	0.1
6.	Bullock cart	4.6	3.2	1.2
7.	Hand cart	4.6	3.2	0.3

The Indian Roads Congress has given set of tentative PCU values or Equivalency Factors for rural road in even sections of plain terrain (Ref. 22) and these are presented in Table 5.6. However the IRC has suggested the set same of tentative Equivalency Factors for use on urban roads also (Ref. 23).

Table 5.6 Tentative Equivalency factors suggested by the IRC

S. No.	Vehicle class	Equivalency Factors
1.	Passenger car, tempo, autorickshaw, agricultural tractor	1.0
2.	Bus, truck, agricultural tractor-tailer unit	3.0
3.	Motor cycle, scooter and pedal cycle	0.5
4.	Cycle rickshaw	1.5
5.	Horse drawn vehicles	4.0
6.	Small bullock cart and hand cart	6.0
7.	Large bullock cart	8.0

Practical Capacity Values

The practical capacity values suggested by the IRC for the purpose of design of different types of roads in rural areas are given in Table 5.7.

Tentative capacity values of urban roads (mid-block sections, between intersections) suggested by the Indian Road Congress are given in Table 5.8.

Table 5.7 Capacity of different types of roads in rural areas

Types of road	Capacity PCU per day (both directions)
Single lane with 3.75 m wide carriageway and normal earthen shoulders	1000
Single lane roads with 3.75 m wide carriageway and 1.0 m wide hard shoulders	2500
Roads with intermediate lanes of width 5.5 m and normal earthen shoulders	5000
Two lane roads with 7.0 m wide carriageway and earthen shoulders.	10,000
Four lanes divided highway (depending on traffic, access control, etc.)	20,000 to 30,000

Table 5.8 Capacity of Urban Roads

No. of Traffic lanes and width	Traffic Flow	Capacity in PCU per hour for traffic condition		
		(i) Roads with no frontage access, no standing vehicles, very little cross traffic	(ii) Roads with frontage access, but no standing vehicle and high capacity intersections	(iii) Roads with free frontage access, parked vehicles & heavy cross traffic
Two lane (7.0 – 7.5)	One way	2400	1500	1200
Two lane (7.0 – 7.5)	Two way	1500	1200	750
Three lane (10.5 m)	One way	3600	2500	2000
Four lane (14.0 m)	One way	4800	3000	2400
Four lane (14.0 m)	Two way	4000	2500	2000
Six lane (21.0 m)	Two way	6000	4200	3600

Example 5.9

Estimate the theoretical capacity of a traffic lane with 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 time = 0.7 sec; assume average length of vehicles = 5.0 m.

Solution

$$V = 40 \text{ kmph}; t = 0.7 \text{ sec}; L = 5.0 \text{ m}$$

$$S = 0.278\, V t + L = 0.278 \times 40 \times 0.7 + 5.0 = 12.78 \text{ m}$$

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

Parking Studies

The demand by automobile users of parking space is one of the major problems of highway transportation, especially in metropolitan cities. In industrial, commercial and residential places with multi-storeyed buildings, parking demand is particularly high. Parking studies are useful to evaluate the facilities available.

Various aspects to be investigated during parking studies are :

(i) *Parking demand* : The parking demand may be evaluate by different methods. One of the methods is by making cordon counts of the selected area and recording accumulation of vehicles during the peak hours by subtracting the outgoing traffic from the traffic volume entering the cordoned area.

One other 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. By noting the registration number of each parked vehicle at any desired time interval (such as 30 minute, one hour, etc.) it is possible to estimate the duration of parking of each vehicle at the parking area. Another useful method of field study is by interviewing the drivers of parked vehicles, shop owners and other vehicle owners in the locality. This method is very useful when the parking demand in the study area is higher than the parking space available.

(ii) *Parking characteristics* : The study is directed to note the present parking practices prevalent in the area under consideration and the general problems in parking. In case of kerb parking, it is also necessary to study the parking pattern, interference to smooth flow of traffic and the accidents involved during parking and unparking operations.

(iii) *Parking space inventory* : The area under study is fully surveyed and a map is prepared showing all places where kerb parking and off-street parking facilities can be provided to meet the parking demands. The traffic engineer has to strike a balance between capacity and parking demands and to design proper facilities for parking. The design of parking facilities is presented in Art. 5.5.

Accident Studies

The problem of accident is very acute in highway transportation due to complex flow patterns of vehicular traffic presence of mixed traffic and pedestrians. Traffic accidents may involve property damages, personal injuries or even casualities. One of the main objectives of traffic engineering is to provide safe traffic movements. Road accident cannot be totally prevented, but by suitable traffic engineering and management measures, the accident rate can be decreased considerably. Therefore the traffic engineer has to carry out systematic accident studies to investigate the causes of accidents and to take preventive measures in terms of design and control. It is essential to analyse every individual accident and to maintain zone-wise accident records. The statistical analysis of accidents carried out periodically at critical locations or road stretches or zones will help to arrive at suitable measures to effectively decrease the accident rates.

The various objectives of the accident studies may be listed as :

(i) to study the causes of accidents and to suggest corrective treatment at potential location,

(ii) to evaluate existing designs,

(iii) to support proposed designs,

(iv) to carry out *before* and *after* studies and to demonstrate the improvement in the problem,

(v) to make computations of financial loss, and

(vi) to give economic justification for the improvements suggested by the traffic engineer.

Causes of accidents

There are four basic elements in a traffic accident :

(i) the road users

(ii) the vehicles

(iii) the road and its condition, and

(iv) environmental factors-traffic, weather etc.

The road user responsible for the accident may be the driver of one or more vehicles involved, pedestrians or the passengers. Vehicles involved in the accident may also be defective. The condition of the road surface or other existing geometric features or any of the environmental conditions of the road may not be upto the expectation causing an accident. To sum up, an accident may be caused due to a combination of several reasons and seldom due to one particular reason. Hence it is often not possible to pin point a particular single cause of an accident.

Various causes of accidents may hence be listed as given below :

(a) *Drivers* : Excessive speed and rash driving, carelessness, violation of rules and regulations, failure to see or understand the traffic situation, sign or signal, temporary effects due to fatigue, sleep or alcohol.

(b) *Pedestrians* : Violating regulations, carelessness in using the carriageway meant for vehicular traffic.

(c) *Passengers* : Alighting from or getting into moving vehicles.

(d) *Vehicle defects* : Failure of brakes, steering system, or lighting system, tyre burst and any other defect in the vehicles.

(e) *Road condition* : Slippery or skidding road surface, pot holes, ruts and other damaged conditions of the road surface.

(f) *Road design* : Defective geometric design like inadequate sight distance, inadequate width of shoulders, improper curve design, improper lighting and improper traffic control devices.

(g) *Weather* : Unfavourable weather condition like mist, fog, snow, dust, smoke or heavy rainfall which restrict normal visibility and render driving unsafe.

(h) *Animals* : Stray animals on the road

(i) *Other causes* : Incorrect signs or signals, gate of level crossing not closed when required, ribbon development, badly located advertisement boards or service stations etc.

Accident studies and records

The various steps involved in traffic accident studies are collection of accident data, preparation of reports, location file and diagrams, and application of the above records for suggesting preventive measures.

(i) Collection of accident data :

The collection of accident data is the first step in the accident study. Standard form for collecting the data are prepared, as suggested by the IRC (see Ref. 25).

The details to be collected are briefly mentioned here.

(a) *General* : Date, time, persons involved in the accident and their particulars, classification of accident like fatal, serious, minor etc.

(b) *Location* : Description and details of the location of accident.

(c) *Details of vehicles involved* : Registration number make and description of the vehicles, loading details, vehicular defects.

(d) *Nature of accident* : Condition of vehicles involved, details of collision, and pedestrians or objects involved, damages, injuries casuality etc.

(e) *Road and traffic conditions* : Details of road geometrics, whether the road is straight or curved, surface characteristics such as dry, wet or slippery etc. Traffic condition – type of traffic, traffic density, etc.

(f) *Primary causes of accident* : Various possible causes and the primary cause of the accident.

(g) *Accident costs* : The total cost of the accident computed in terms of rupees, of the various involvements like property damages, personal injuries and casualities.

(ii) Accident report :

The accident should be reported to police authorities who would take legal actions especially in more serious accidents involving injuries, casualities or severe damage to property. Accident report of the individuals involved may be separately taken. The accident data should be collected as given above and the accident report is prepared with all facts which might be useful in subsequent analysis, claims for compensation , etc.

(iii) Accident records :

The accident records are maintained giving all particulars of the accidents, location other details. The records may be maintained by means of location files, spot maps, collision diagrams and condition diagrams.

(a) *Location files* : These are useful to keep a check on the location of accident and to identify points of high accident incidence. Location fields should be maintained by each police station for the respective jurisdiction.

(b) *Spot maps* : Accident location spot maps show accidents by spots, pins or symbols on the map. A map of suitable scale say, 1 cm = 40 to 60 metre, may be used for spotting urban accidents. The common legend used for spot maps, are given in Fig. 5.12.

Fig. 5.12 Legend for Spot Maps

(c) *Condition diagram* : A condition diagram is a drawing to scale showing all important physical conditions of an accident location to be studied. The important features generally to be shown in this diagram with suitable dimensions marked there in an roadway limits, curves, kerb lines, bridges, culverts trees and all details of roadway conditions, obstruction to vision, property lines, signs, signals etc. There are standard symbols used in showing various details. The condition and collision diagrams may be combined together in a single sketch, if necessary.

(d) *Collision diagram* : These are diagrams showing the approximate path of vehicles and pedestrians involved in the accidents. Collision diagrams are most useful to compare the accident pattern before and after the remedial measures have been taken.

A typical collision diagram and symbols are shown in Fig. 5.13.

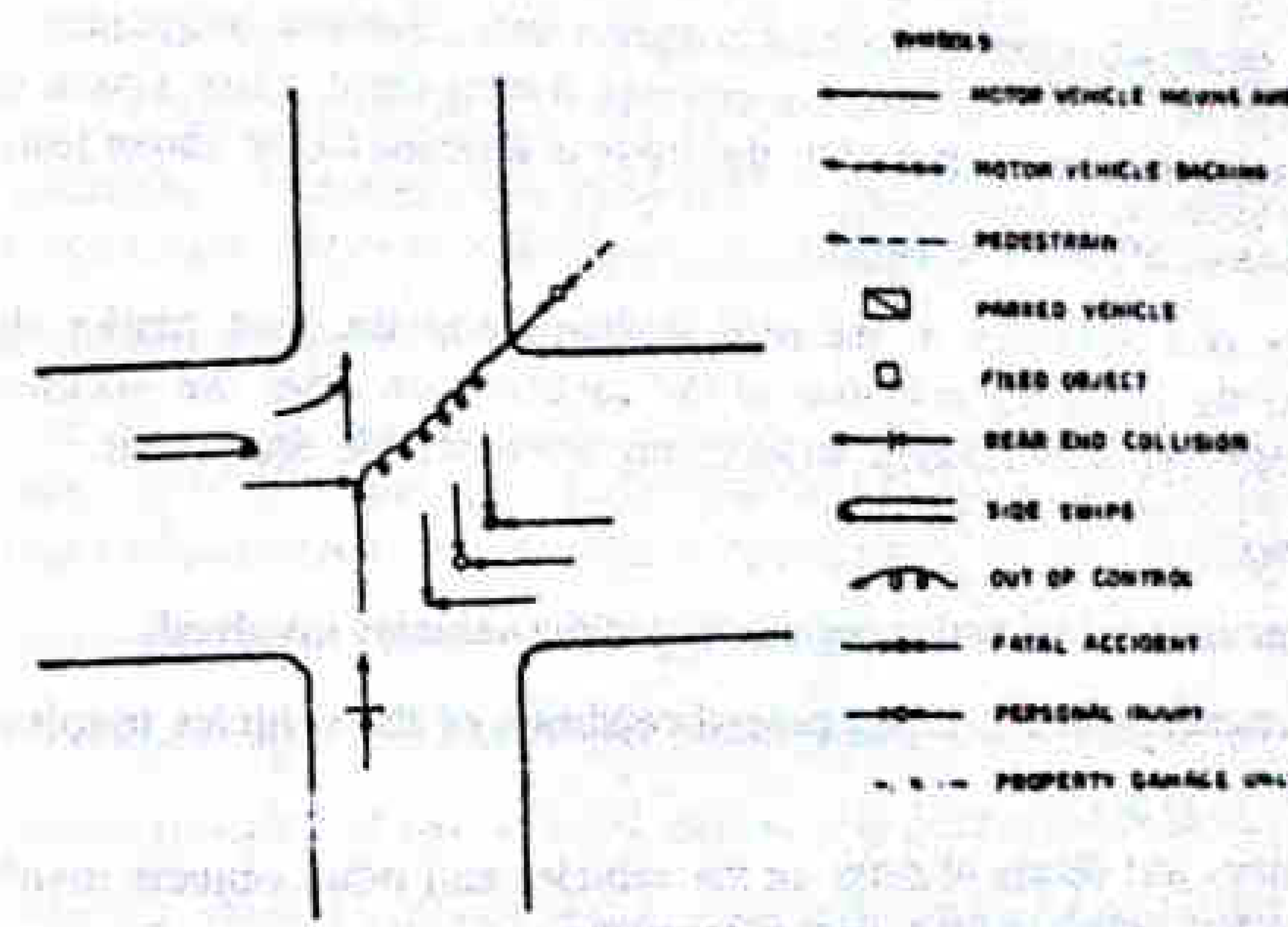


Fig. 5.13 Collision Diagram and Symbols

Accident investigations and studies therefore may be carried out scientifically in the following three stages :

Accident Investigations

Analysis of Individual Accidents

Statistical Analysis of Accidents

Accident investigations

The scientific approach for accident investigations suggested by the authors are summarized below. It is suggested that a mobile laboratory may be kept ready in every city. A bus equipped with essential instruments to measure the alcohol content in the breath, reaction time and other driver characteristics, skid resistance of pavement surface, etc, and a traffic engineer and his assistants may form the proposed mobile laboratory which should reach the accident spot as soon as possible after an accident. The following investigations may be carried out to enable analysis of the accident on a scientific basis.

(i) Recording General Observations

(a) Measurement of length of skid marks due to partial and full skidding.

(b) Recording the relative positions of vehicles and objects involved in the accident and collision diagram supplemented with photographs.

(c) Details of accident, injuries and damages

(d) Condition of pavement surface, shoulders and other surface through which the vehicles involved in the accident have moved; environmental conditions.

(e) Conditions diagram of accident locations with relevant measurements and dimensions.

(ii) Driver Tests

(a) Analysis of breath of drivers involved in the accident for alcohol content (using a suitable breathalyzer; if alcohol consumption is indicated above a prescribed limit, collection of blood sample for further analysis in a forensic laboratory). In case the driver is dead, tests may be conducted on the spinal fluid for estimating the alcohol content, if any,

(b) Tests on driver characteristics such as reaction time, distance judgement, angle of clear vision etc. If the accident has occurred during night, glare vision and glare recovery tests are to be conducted on the driver in addition to the above tests.

(iii) Skid Resistance of Pavement Surface

The average skid resistance of the road surface along the skid marks should be measured under the prevailing condition of the surface soon after the accident, using portable pendulum type skid resistance tester or any other suitable equipment.

(iv) Vehicle Tests

(a) Tests on the condition of brakes and steering of the vehicles involved.

(b) Tests on essential accessories and general condition of the vehicles involved in the accident.

(c) Characteristics and details of dents on the vehicles and other objects involved and the cross section details of the collapsed members.

(v) Probable Causes of the Accident

Assessment of the probable causes (primary, secondary and contributing causes) of the accident, its type, site conditions, position of the vehicles and other objects involved and other existing conditions.

(vi) Cost Analysis

Estimation of the cost of accident by working out the cost involved for the following items :

(a) Injuries and fatalities of persons involved

(b) Damages to the vehicles

(c) Property damages

(d) Other consequences including traffic delay

(e) Investigations and legal proceeding

Statistical analysis of accidents

The statistical analysis of road accidents help to assess the effectiveness of various measures to decrease the accident rate ; thus the analysis estimates the relative safety of road stretches or zones.

As the mobility increases the probability of accident also increases. The number of accidents are found to increase with the number of road users or the number of vehicles and pedestrians on the roads. As the vehicle movements and the population are on the increase, the total number of accidents in the study area are likely to increase year after year. The effectiveness of traffic engineering, enforcement and educational measures may therefore be judged from the changes in the annual accident rates, rather the total number.

The accident rate may either be expressed in terms of the number of vehicles and population or the vehicle movements (in vehicle-km) and the population. Relative accident rate may also be expressed in terms of various factors of the roadway and traffic. Accident-prone stretches of different roads may be assessed by finding the accident density per unit length of road. By statistical study of accident occurrence at a particular road or location or a zone of study for a long period of time, it is possible to predict with reasonable accuracy the probability of accident occurrence per day or the relative safety of different classes of road users in that location.

The reliability of the statistical analysis of accidents depends mainly on the reliability of the accident data, obtained from the accident records.

Analysis of individual traffic accidents

Each of the road accidents is analysed by the traffic engineer to draw sound conclusions. It is necessary to compute the original speeds of the vehicles involved in various types of accidents. Some of the typical modes of vehicular accidents are :

(i) A moving vehicle collides with a parked vehicle

(ii) Two vehicles approaching from different directions collide at an intersection

(iii) Head-on collision of two vehicles approaching from opposite directions

(iv) A moving vehicle collides with a stationary object like an electric pole, tree or a rigid structure.

The method of analysis for accident type (i) and (ii) mentioned above have been given below, as these are quite common type of accidents.

In order to simplify the analysis, some assumptions are made as discussed below :

(i) When skid marks are present, the skid distances are measured to find the actual braking distances and it is assumed that 100 percent skid has occurred. When skid marks are not at all visible, it may be assumed as a free collision, without the brakes being applied.

(ii) When two vehicles of masses m_a and m_b with speeds v_a and v_b collide, if it is assumed that both are perfectly plastic bodies, both would move together with the same speed v′ after impact and the relationship is given by :

$$(m_a v_a + m_b v_b) = (m_a + m_b) v'$$

If both the bodies are perfectly elastic, the coefficient of restitution will be unity and relationship is given by :

$$(v_A - v_B) = (v_B' - v_A')$$

In case the coefficient of restitution e is known, then the relation is :

$$e\,(v_A - v_B) = (v_B' - v_A')$$

The actual values of the coefficient of restitution should be either known or suitably assumed.

(iii) The impact of the vehicles may be either direct or oblique, at a known angle.

(iv) The friction coefficient of the pavement surface under the prevailing conditions may either be determined from field test or be suitably assumed. However the friction coefficient is assumed to be uniform throughout the skid.

Analysis of speed from skid distance

The basic equation for finding the braking distance or skid distances S for a vehicle of weight W to slow down from speed v_1 to v_2 m/sec is obtained by equating the reduction in kinetic energy with the work done against the frictional force i.e.,

$$\frac{W}{2g}\left(v_1^2 - v_2^2\right) = W.f.S$$

where f is the average friction factor or skid resistance developed.

$$S = \frac{v_1^2 - v_2^2}{2gf} \tag{5.4}$$

Substituting the values of g = 9.8 m/sec^2 and the speed in V_1 and V_2 kmph,

$$S = \frac{V_1^2 - V_2^2}{254\,f} \tag{5.5}$$

If the skid distance S is measured from the skid marks, the initial speed v_1 may be calculated from the relation

$$v_1 = \sqrt{v_2^2 + 2g\,f\,S} \tag{5.6}$$

In kmph units,

$$V_1 = \sqrt{v_2^2 + 254\,f\,S} \tag{5.7}$$

If the vehicle comes to a stop after the skid distance S, then v_2 would be zero in this equation.

Case (i)

Collision of moving vehicle with parked vehicle

Suppose a vehicle A, moving with speed v_1 m/sec skids through a distance S_1 after the application of the brakes, collides with a parked vehicle B and the two vehicles skid together through a distance S_2 before coming to a stop. The problem is to estimate the initial speed of vehicle A, v_1 m/sec or V_1 kmph.

(a) *Before collision* the vehicle A of weight W_A moving with initial speed v_1 m/sec applies brakes, skids through a distance S_1 and attains a speed v_2 m/sec just before collision. From Equation 5.6 the relationship between v_1 and v_2 is given by,

$$v_1^2 = v_2^2 + 2g\,f\,S_1 \tag{5.8a}$$

(b) *At collision* with stationary vehicle B of weight W_B, both start moving together at speed v_3 m/sec. Here a perfectly plastic impact is assumed with e = 0. The relation between v_2 and v_3 is given by the momentum equation

$$\frac{W_A}{g}\,.\,v_2 = \frac{W_A + W_B}{g}\,.\,v_3 \text{ or } v_2 = \frac{W_A + W_B}{W_A}\,.\,v_3$$

Substituting in Eq. 5.8a

$$v_1^2 = \left(\frac{W_A + W_B}{W_A}\right)^2 v_3^2 + 2g\,f\,S_1 \tag{5.8b}$$

(c) After collision vehicles A and B skid through distance S_2 before coming to a stop (velocity v_3 reducing to $v_4 = 0$). The relation of v_3 in terms of S_2 is obtained again from Eq. 5.9 :

$$v_3^2 = 2g\,f\,S_2$$

Substituting v_3 in Eq. 5.8b,

$$v_1^2 = \left(\frac{W_A + W_B}{W_A}\right)^2 2g\,f\,S_2 + 2g\,f\,S_1$$

$$v_1 = \sqrt{\left(\frac{W_A + W_B}{W_A}\right)^2 2g\,f\,S_2 + 2g\,f\,S_1} \tag{5.9}$$

In kmph units,

$$V_1 = \sqrt{254\,f\left[S_2\left(\frac{W_A + W_B}{W_A}\right)^2 + S_1\right]} \tag{5.10}$$

where W_A = weight of moving vehicle, kg

W_B = weight of parked vehicle, kg

f = average friction coefficient

S_1 = initial skid distance before collision, metre

S_2 = skid distance of both the vehicles together after collision, metre.

When the vehicle A does not apply brakes and does not skid before collision,

S_1 = 0 in Equation 5.9 and 5.10

Example 5.10

A vehicle of weight 2.0 tonne skids through a distance equal to 40 m before colliding with another parked vehicle of weight 1.0 tonne. After collision both the vehicles skid through a distance equal to 12 m before stopping.

Compute the initial speed of the moving vehicle. Assume coefficient of friction as 0.5.

Solution

Method (i) By Steps :

This problem may be solved easily in three steps without using the Eq. 5.9 or 5.10.

Let the original speed of the vehicle be v_1 m/sec., reduced to v_2 m/sec by applying brakes and skidding through s_1 = 40 m; just after the collision, let both vehicles A and B start moving together with speed v_3 m/sec and finally stop, $v_4 = 0$, after skidding through distance s_2 = 12 m, f = 0.5.

(a) After collision

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

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

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

or $$v_3 = \sqrt{2 \times 9.8 \times 0.5 \times 12} = \sqrt{117.6} \text{ m/sec}$$

(b) At collision

Momentum before impact = momentum after impact

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

$$\frac{W_a + W_b}{W_a} = \frac{2+1}{2} = \frac{3}{2}$$

$$v_2 = \frac{(W_a + W_b)}{W_a} v_3 = \frac{3}{2}\sqrt{117.6} \text{ m/sec}$$

(c) Before collision

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

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

$$v_1^2 = 2g\, f\, s_1 + v_2^2 = 2 \times 9.8 \times 0.5 \times 40 + \frac{9}{4} \times 117.6$$

$$= 656.6$$

$$v_1 = 25.6 \text{ m/sec}$$

$$\text{Original speed } V_1 = 3.6 \times 25.6 = 92.2 \text{ kmph}$$

Method (ii) By using Equation 5.10

$$V_1 = \sqrt{254\, f\left[s_2\left(\frac{W_a + W_b}{W_a}\right)^2 + s_1\right]}$$

$$= \sqrt{254 \times 0.5\left[12\left(\frac{3}{2}\right)^2 + 40\right]} = 92.2 \text{ kmph}$$

Case (ii)

Two vehicles approaching from right angles collide

Two vehicles A and B on approaching an intersection are assumed to skid on application of brakes; they collide with each other and skid further in different directions as illustrated in Fig. 5.14 (a), (b) and (c). The direction of the skidding vehicles after collision in this case depends on the initial speeds of the two vehicles and their weights.

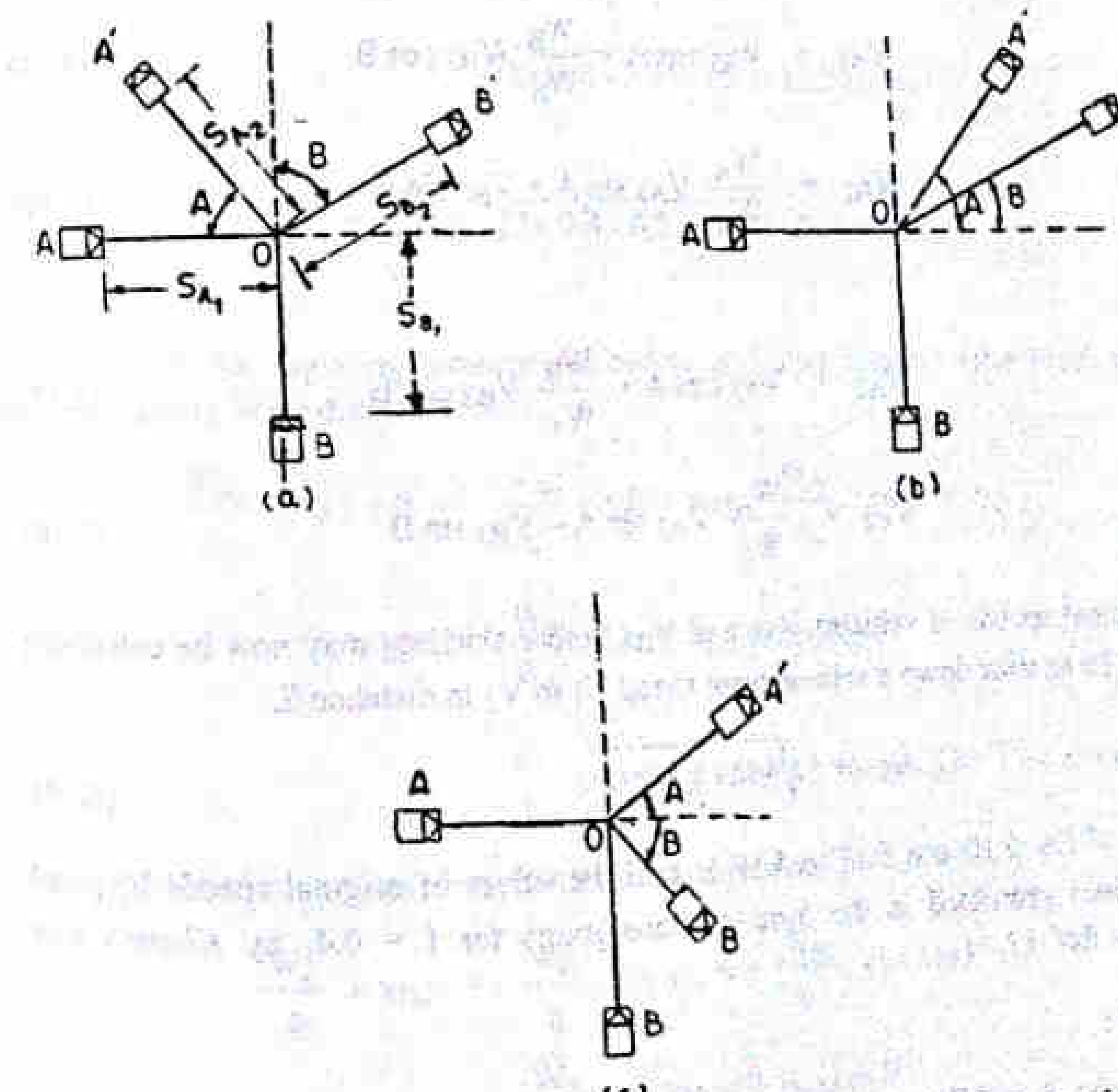


Fig. 5.14 Collision of Two Vehicles approaching from Right Angles

If S_{A2} and S_{B2} are the skid distance of the vehicles after the collision (Fig. 5.14) the speeds of vehicles V_{A3} and V_{B3} just after collision may be found from relations :

$$V_{A3} = \sqrt{254\,f\,S_{A2}} \quad (5.11)$$

$$V_{B3} = \sqrt{254\,f\,S_{B2}} \quad (5.12)$$

The momentum of the vehicles just after collision may be found using the speed values from Eq. 5.11 and 5.12 and these resolved in the original direction of motion of the two vehicles. As per the assumption, the momentum before collision is taken equal to the momentum after collision. For the three cases of right angle collision shown in Fig. 5.14, the speeds of the vehicles just before collision, but after skidding through distances S_{A1} and S_{B1} i.e. V_{A2} or V_{B2} are obtained by the relations given below for the three cases :

Case (a)

$$V_{A2} = \frac{W_{B2}}{W_A} V_{B3} \sin B - V_{A3} \cos A \quad (5.13)$$

$$V_{B2} = \frac{W_A}{W_B} V_{A3} \sin A + V_{B3} \cos B \quad (5.14)$$

Case (b)

$$V_{A2} = V_{A3} \cos A + \frac{W_B}{W_A} V_{B3} \cos B \quad (5.15)$$

$$V_{B2} = \frac{W_A}{W_B} V_{A3} \sin A + V_{B3} \sin B \quad (5.16)$$

Case (c)

$$V_{A2} = V_{A3} \cos A + \frac{W_B}{W_A} V_{B3} \cos B \quad (5.17)$$

$$V_{B2} = \frac{W_A}{W_B} V_{A3} \sin A - V_{B3} \sin B \quad (5.18)$$

The original speeds of vehicles V_{A1} and V_{B1} before skidding may now be calculated using Eq. 5.10 to slow down a vehicle from speed V_1 to V_2 in distance S.

i.e.,

$$V_1 = \sqrt{254\,f\,S_1 + V_2^2} \quad (5.19)$$

Solution of Eq. 5.10 and 5.13 to 5.19 to find the values of original speeds V_{A1} and V_{B1} have been presented in the form of nomograms for f = 0.4, by *Khanna and co-authors* in Ref. 17.

Example 5.11

Two vehicles A and B approaching at right angles, A from West and B from South, collide with each other. After the collision, vehicle A skids in a direction 50° North of West and vehicle B, 60° East of North. The initial skid distances of the vehicles A and B are 38 and 20 m respectively before collision. The skid distances after collision are 15 and 36 m respectively. If the weights of vehicles B and A are 6.0 and 4.4 tonnes, calculate the original speeds of the vehicles. The average skid resistance of the pavement is found to be 0.55.

Solution

Method (i) By Steps

Let the initial speeds of vehicles A and B before brake application be v_{A1} and v_{B1}, the speeds just before collision, after skidding through S_{A1} = 38 and S_{B1} = 20 m be v_{A2} and v_{B2}, the speeds just after collision, be v_{A3} and v_{B3} and the final speed when the vehicles come to a stop is zero; after skidding through further distance S_{A2} = 15 and S_{B2} = 36 m; f = 0.55 (Refer Fig. 5.14 a).

(a) After collision

Loss in kinetic energy of each vehicle = work done against skid resistance

For vehicle A,
$$\frac{W_A\, v_{A3}^2}{2g} = W_A\, f.\, S_{A2}$$

$$\therefore v_{A3} = \sqrt{2g\,f\,S_{A2}}$$

$$= \sqrt{2 \times 9.8 \times 0.55 \times 15} = 12.7 \text{ m/sec}$$

Similarly,

$$V_{B3} = \sqrt{2 \times 9.8 \times 0.55 \times 36} = 19.7 \text{ m/sec}$$

(b) At collision

From Fig. 5.14a, equating momentums before and after impacts after resolving the momentums along West-East direction,

$$\frac{W_A}{g} \times v_{A2} + 0 = \frac{W_B}{g} \sin B \times v_{B3} - \frac{W_A}{g}.\cos A \times v_{A3}$$

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

$$= \frac{6}{4}.\sin 60 \times 19.7 - 12.7 \times \cos 50 = 17.4 \text{ m/sec}$$

Resolving the momentums along South-North direction,

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

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

$$= \frac{4}{6} \times 12.7 \times \sin 50 + 19.7 \cos 60 = 16.4 \text{ m/sec}$$

(c) After collision

Loss in kinetic energy due to brake application = work done against brake application

i.e., $$\frac{W_A}{2g}(v_{A1}^2 - v_{A2}^2) = W_A . f . S_{A1}$$

$$\therefore v_{A1}^2 = 2g\,f\,S_{A1} + v_{A2}^2$$

$$= 2 \times 9.8 \times 0.55 \times 38 + 17.4^2 = 712.4$$

$$v_{A1} = 26.7 \text{ m/sec}, V_{A1} = 26.7 \times 3.6 = 96 \text{ kmph}$$

Similarly:

$$V_{B1}^2 = 2g\,f\,S_{B1} + v_{B2}^2 = 2 \times 9.8 \times 0.55 \times 20 + 16.4^2$$

$$= 484.6$$

$$v_{B1} = 22 \text{ m/sec}, V_{B1} = 22 \times 3.6 = 79.2 \text{ kmph}$$

Method (ii) By using the equations

Using Eq. 5.11 and 5.12, speeds of vehicles just after collision,

$$V_{A3} = \sqrt{254\,f\,S_{A2}} = \sqrt{254 \times 0.55 \times 15} = 45.8 \text{ kmph}$$

$$V_{B3} = \sqrt{254 \times 0.55 \times 36} = 70.9 \text{ kmph}$$

Using Eq. 5.13 and 5.14, speeds of vehicles just before collision,

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

$$= \frac{6}{4} \times 70.9 \times \sin 60 - 45.8 \cos 50 = 92.1 - 29.4$$

$$= 62.7 \text{ kmph}$$

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

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

$$= 23.4 + 35.5 = 58.9 \text{ kmph}$$

Original speeds of vehicles before application of brakes are obtained using Eq. 5.19.

$$V_{A1} = \sqrt{254\,f\,S_{A1} + V_{A2}^2}$$

$$= \sqrt{254 \times 0.55 \times 38 + 62.7^2} = 96 \text{ kmph}$$

$$V_{B1} = \sqrt{254\,f\,S_{B1} + V_{B2}^2}$$

$$= \sqrt{254 \times 0.55 \times 20 + 58.9^2} = 79 \text{ kmph}$$

Thus by both the methods, the original speeds of vehicles A and B before the application of brakes are 96 and 79 kmph respectively.

Measures for the reduction in accident rates

The various measures to decrease the accident rates may be divided into three groups :

(i) Engineering

(ii) Enforcement

(iii) Education

These three measures are generally termed "3-E_s". The details of these measures are given below.

Engineering Measures

(a) Road design : The geometric design features of the road such as sight distances, width of pavement, horizontal and vertical alignment design details and intersection design elements are checked and corrected if necessary. The pavement surface characteristics including the skid resistance values are checked and suitable maintenance steps taken to bring them upto the design standards. Where necessary by-passes may be constructed to separate through traffic from local traffic. To minimise delay and conflicts at the intersections, it may be essential to design and construct grade separated intersections or fly overs.

(b) Preventive maintenance of vehicles : The braking system, steering and lighting arrangements of vehicles plying on the roads may be checked at suitable intervals and heavy penalties levied on defective vehicles. These measures are particularly necessary for public carriers.

(c) Before and after studies : The record of accidents and their patterns for different locations are maintained by means of collision and condition diagrams. After making the necessary improvements in design and enforcing regulation, it is again necessary to collect and maintain the record of accidents "before and after" the introduction of preventive measures to study their efficiency. A typical example of before and after study at an intersection is shown in Fig. 5.15.

(d) Road lighting : Proper road lighting can decrease the rate of accidents during night, due to poor visibility. Lighting is particularly desirable at intersections, bridge sites and at places where there are restrictions to traffic movements.

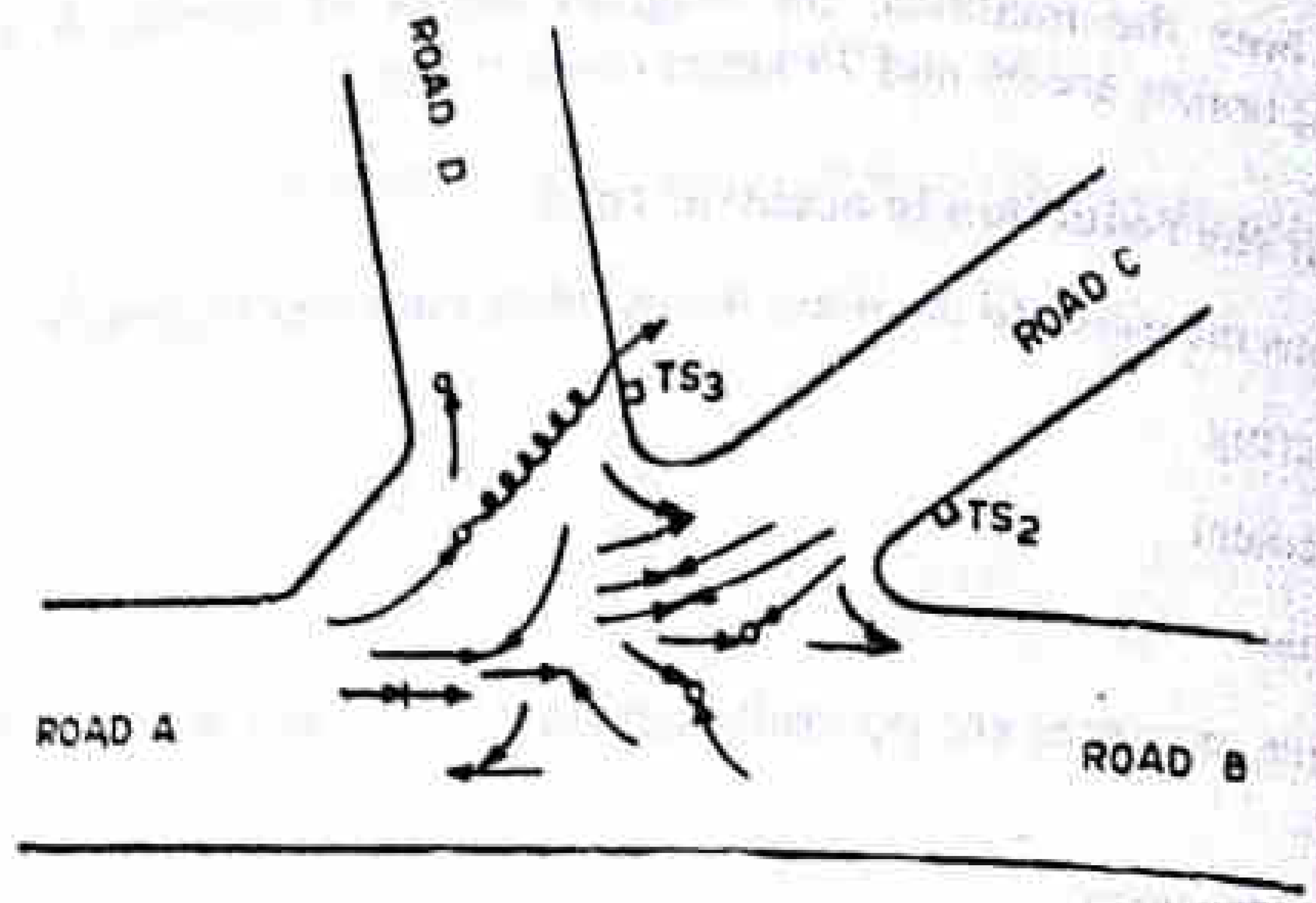
Enforcement Measures

The various measures of enforcement that may be useful to prevent accidents at spots prone to accidents are enumerated here. The motor vehicle rules are revised from time to time to make them more comprehensive.

(a) *Speed control* : To enable drivers of buses to develop correct speed habits tachometers may be fitted so as to give the record of speeds. Also surprise checks on spot speed of all fast moving vehicles should be done at selected locations and timings and legal actions on those who violate the speed limits should be taken.

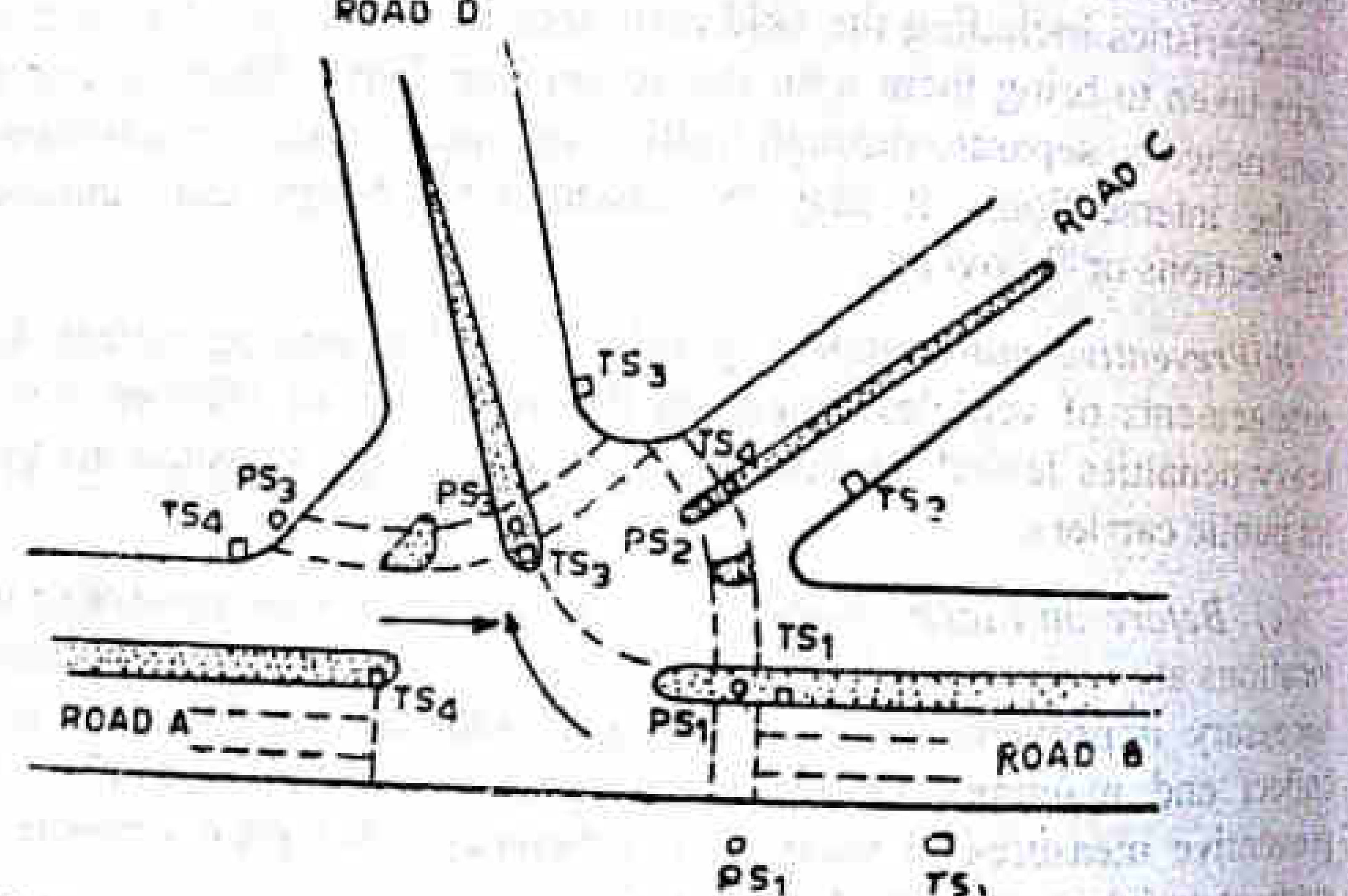
(b) Traffic control devices : Signals may be re-designed or signal system be introduced if necessary. Similarly proper traffic control device like signs, markings or channelizing islands may be installed wherever found necessary.

(a) Before Study



Uncontrolled movements of vehicles and pedestrians
Accidents : Twelve in a period of two months

(b) After Study



(i) Additional traffic signals (T.S.) installed and signal timings re-designed
(ii) Pedestrian signals (P.S.) installed and pavement markings made for pedestrian crossings and control over other vehicular manoeuvers.
(iii) Divisional islands and channelizing islands provided by widening roads A-B.
Accidents : only on in a period of two months

Fig. 5.15 Typical Case of Before and After Study

(c) *Training and supervision* : The transport authorities should be strict in testing and issuing license to drivers of public service vehicles and taxis. Even the drivers who have passed the requisite tests should be kept under proper supervision and be trained in proper defensive driving. Driving license of the driver may be renewed after specified period, only after conducting some tests to check whether the driver is fit.

(d) *Medical check* : The drivers should be tested for vision and reaction time at prescribed intervals, say, once in three years.

(e) *Special precautions for commercial vehicles* : It may be insisted on having a conductor or attendant to help and give proper direction to drivers of heavy commercial vehicles.

(f) *Observance of law and regulation* : This is one of the most essential steps in enforcement for prevention of accidents. Traffic or transport authorities should send study groups of trained personnel, assisted by police to different locations to check whether the traffic regulations are being followed by the road users and also to enforce the essential regulations. The study group can provide useful data for deciding about the necessity of revision of certain traffic regulations.

Educational Measures

(a) *Education of road users* : It is very essential to educate the road users for the various precautionary measures to use the road way facilities with safety. The passengers and pedestrians should be taught the rules of the road, correct manner of crossing etc. This may be possible by introducing necessary instruction in the schools for the children. Posters exhibiting the serious results due to carelessness of road users may also be useful. The Indian Roads Congress has recently prepared Highway Safety Code and the document on Road Safety for school children and an Instruction Manual on Road Safety Education is under preparation.

(b) *Safety drive* : Imposing traffic safety week when the road users are properly directed by the help of traffic police and transport staff is a common means of training the public these days. Roads users should be impressed on what should and what should not be done, with the help of films and documentaries. Training courses may be conducted for drivers. The IRC has been orgainsing Highway Safety Workshop in different regions of the country.

5.2.4 Relationship Between Speed, Travel Time, Volume, Density and Capacity

In the operation and planning of traffic facilities the relationship between the fundamental stream flow variables is important.

The *travel time* per unit length of road is inversely proportional to the speed. If T is travel time and V is the speed (kmph).

$$T\ (\text{min/km}) = \frac{60}{V} \tag{5.20a}$$

or

$$T\ (\text{sec/km}) = \frac{3600}{V} \tag{5.20b}$$

Figure 5.16 shows the relationship between travel time and speed. It is seen that at higher speeds, the rate of saving in travel time decreases.

The fundamental relationship between traffic volume, density and speed may be given by the general equation of traffic flow :

$$q = K\,V_s \tag{5.21}$$

where

q = the average volume of vehicles passing a point during a specified period of time; (vehicles per hour)

K = the average density or number of vehicles occupying a unit length of roadway at a given instant (vehicles/km)

V_s = space-mean speed of vehicles in a unit roadway length (kmph)

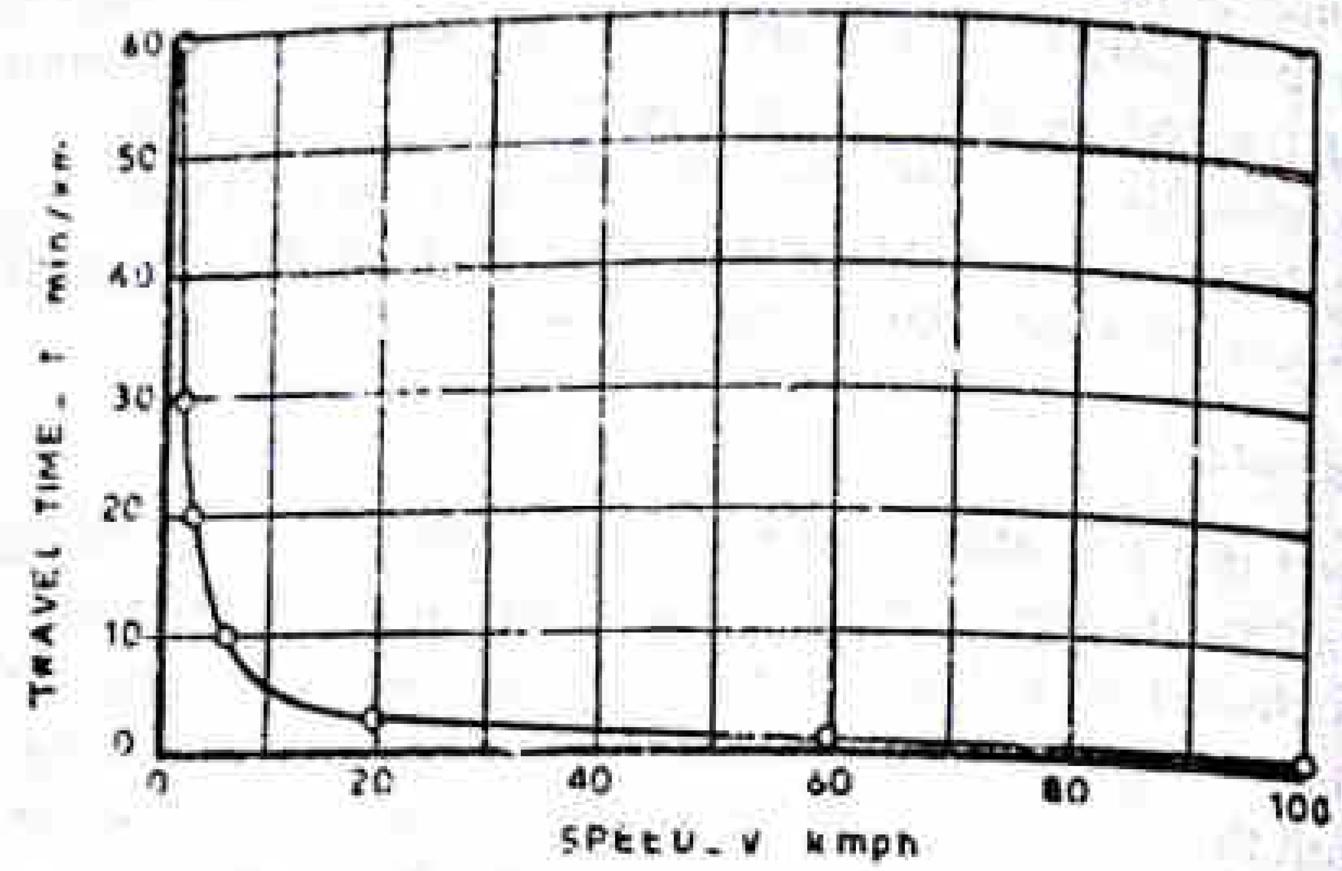


Fig. 5.16 Travel Time and Speed

With increase in speed of vehicles in a unit roadway length the average density decreases. This is because the spacing between the vehicles increases with increase in speed, as discussed in Art. 5.2.3. Field observations of speed and density made simultaneously, have indicated that approximately straight line relationship between speed and density could be obtained for a good range of speeds, particularly when the speed is not high. As the speed approaches zero i.e., towards stand-still maximum density is obtained. Figure 5.17 illustrates a hypothetical case based on the simple model of straight line relationship between speed and density. The dashed line shows the actual trend of observations at higher speeds and the extension by the dotted lines is the hypothetical case.

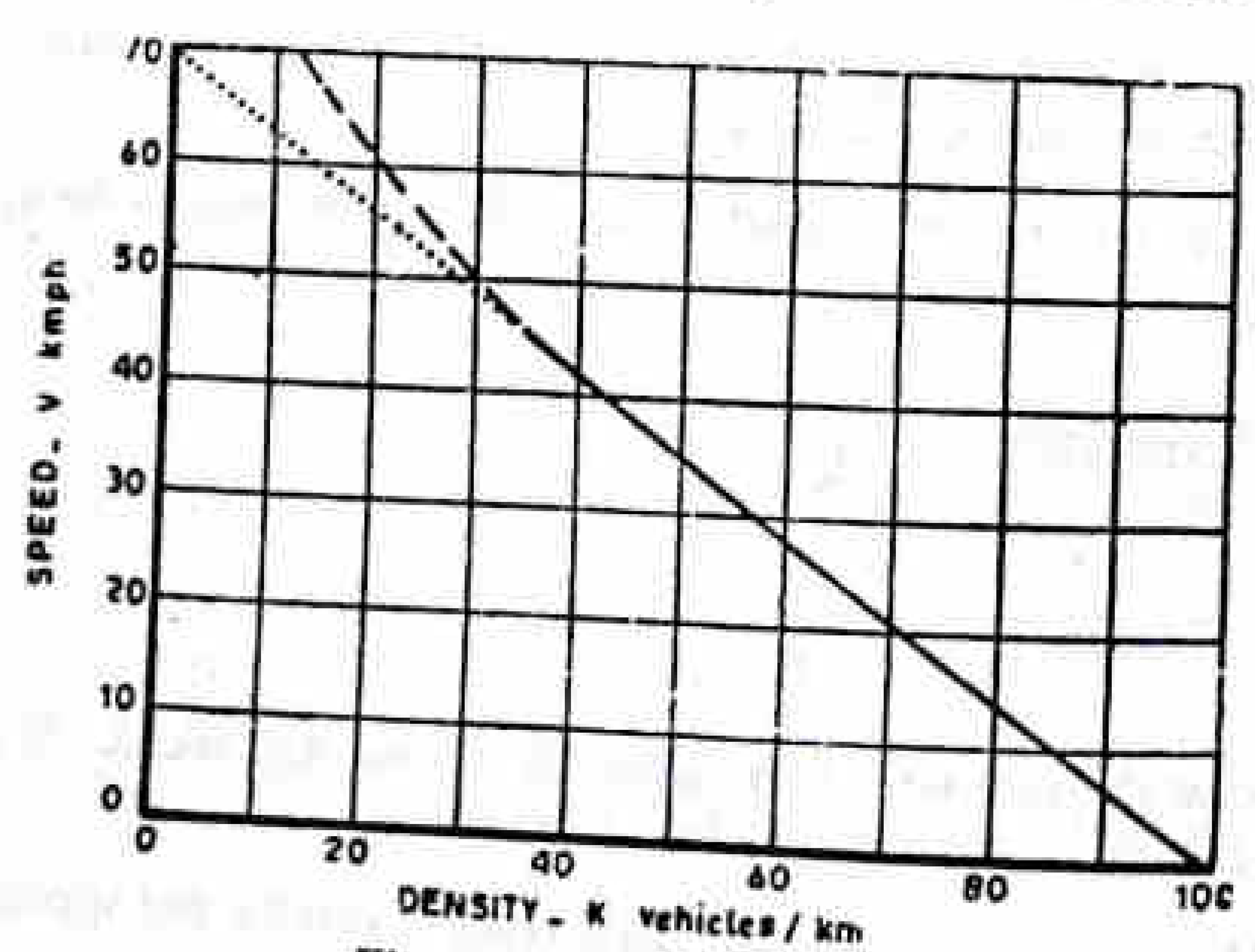


Fig. 5.17 Speed and Density

It is difficult to measure density directly, in practice. Hence the relationship between volume, density and speed (Eq. 5.5) is often used. The value of density K may be obtained by rewriting Eq. 5.5.

$$K \text{ (vehicles/km)} = \frac{q \text{ (vehicles/hr)}}{v \text{ (km/hr)}} \tag{5.22}$$

It is obvious that at very low speeds, the traffic volume would also be low; zero speed indicates zero flow or volume. With increasing speeds, traffic volume also increases upto

a certain limit as the time head-way H_t initially decreases. But as the speed further increases, the spacing between the vehicles becomes so large that the time headway between the vehicles also increases and thus the volume decreases. The relationship between speed and volume of traffic for a hypothetical case is shown in Fig. 5.18. It may be seen that with each value of volume there are two speed values. The dashed line indicates the trend based on observation. Extension by the dotted line is based on the simplified hypothesis.

When the speed of the traffic flow decreases and becomes zero, the density attains the maximum value whereas, volume becomes zero. For increasing values of speeds, density decreases, whereas the volume increases upto a certain limit (Fig. 5.17 and 5.18). At high speeds, the volume starts decreasing and density keeps on further reducing. Eventually if a hypothetical case is considered when volume approaches zero at very high speeds, the density also approaches zero as shown in Fig. 5.19. Thus there is a maximum flow in road corresponding to some optimum values of speed and density.

In Eq. 5.21 if any two of the three stream variables i.e., volume, speed and density, are known, the third may be determine. The traffic volume and speed may be measured easily in the field, whereas it is difficult to measure the density. Hence by measuring the values of traffic volume and speed, it is possible to compute the density. When the three relations given in Fig. 5.17, 5.18 and 5.19 are combined and plotted on three mutually perpendicular axes, the surface obtained may be visualized as the basic traffic stream equation as illustrated in Fig. 5.20.

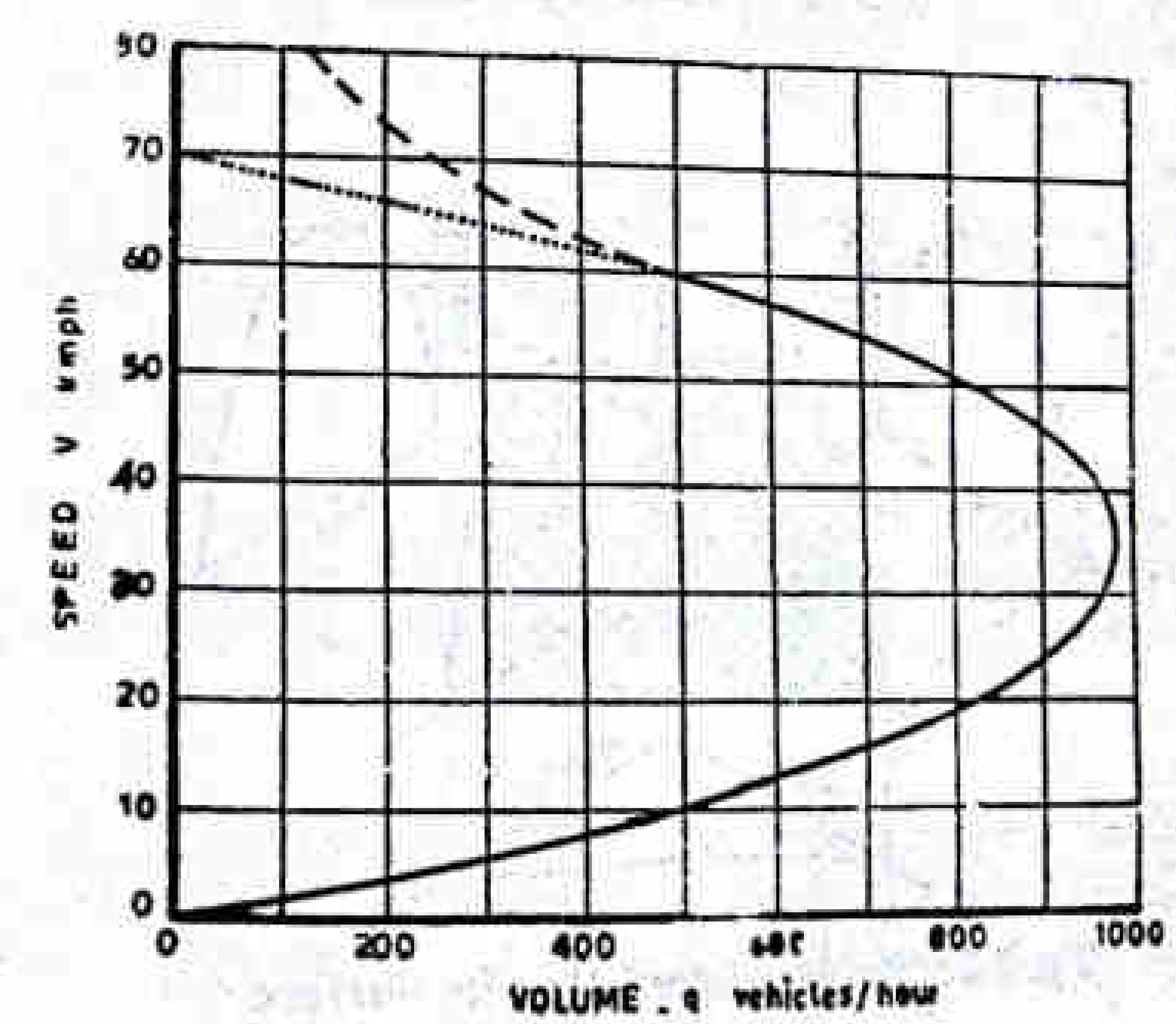


Fig. 5.18 Speed and Volume

The above flow relationships have been established for homogeneous traffic flow. In the case of mixed traffic flow with heterogeneous traffic, these relationships are likely to be quite complex.

Capacity flow

The maximum speed value in Fig. 5.17 and 5.18 is called free means speed V_{sf} and the maximum density at zero speed is called jam density K_j. The maximum flow q_{max} or the capacity flow q_c (see Fig. 5.11) occurs when the speed is $\frac{V_{sf}}{2}$ and density is $\frac{K_j}{2}$ and therefore from Eq. 5.21.

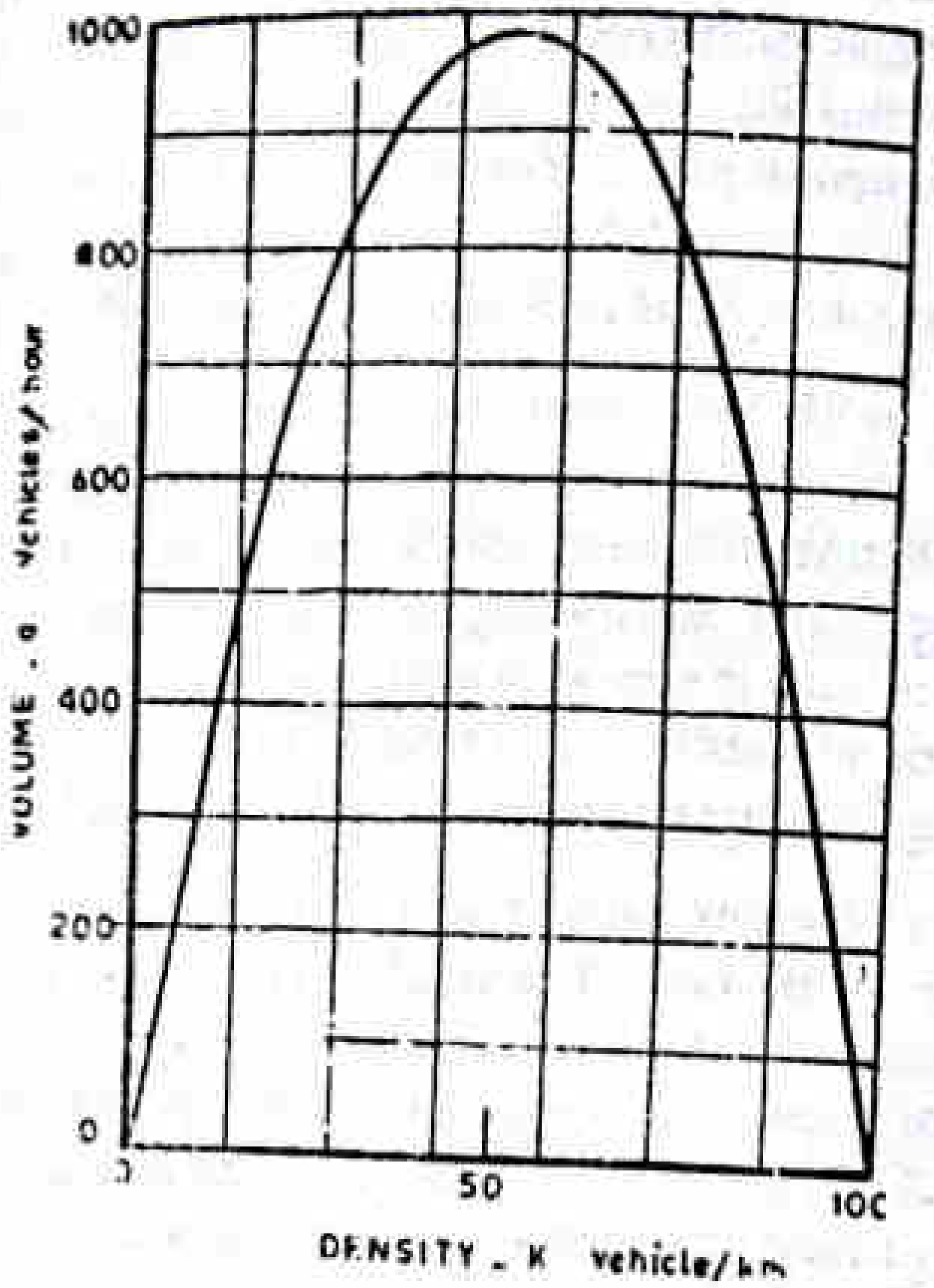


Fig. 5.19 Volume and Density

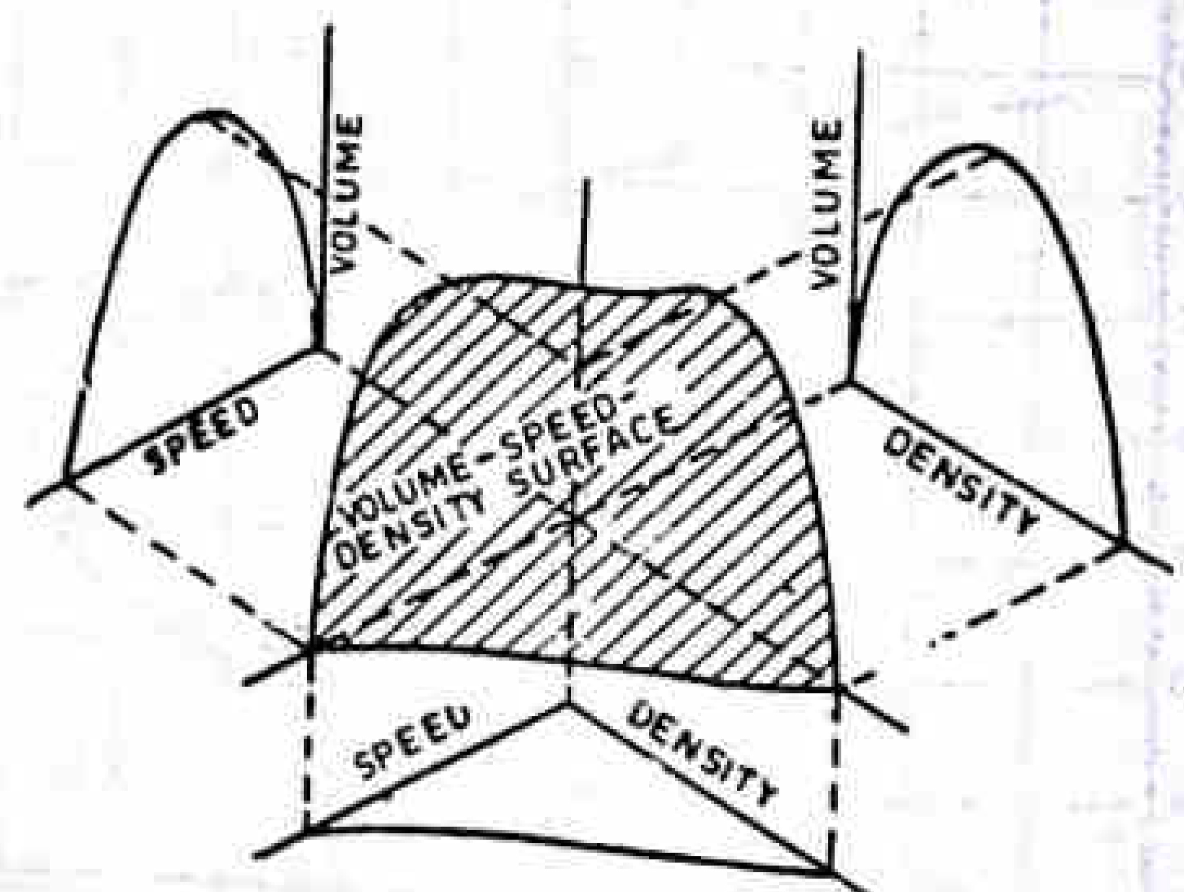


Fig. 5.20 Volume-Speed-Density Surface

$$q_{max} = \frac{V_{sf} \cdot K_j}{4} \tag{5.23}$$

Example 5.12

The free mean speed on a roadway is found to be 80 kmph. Under stopped condition the average spacing between vehicles is 6.9 m. Determine the capacity flow.

Solution

Free mean speed $V_{si} = 80$ kmph

Jam density $K_j = \frac{1000}{6.9} = 145$ vehicles/km (per lane)

Maximum flow $q_{max} = \frac{80 \times 145}{4} = 2900$ vehicles/hour (per lane)

5.2.5 Future Traffic

The existing traffic operation may be measured by traffic studies as described in Art. 5.2.3. It is necessary to know the future traffic demand in order to design highway facilities adequately. There are different methods for projecting future traffic. These may be broadly classified as : (i) mechanical and (ii) analytical methods.

In the mechanical methods, the past trends of traffic growth are simply projected forward, making use of a percentage increase in various categories of traffic in a region. In some countries it has been estimated that the traffic is doubled in a period of 10 years resulting in the annual growth rate of 7 to 8 percent. In mechanical methods any of the principles such as a correlation index, ratio, analogy composite trend or a growth formula may be used. Correlation index may include gross productivity, national income or fuel consumption. A straight line, geometric progression, compound interest or general growth rate may be considered in the growth formulae.

The analytical methods consider the short term as well as long term policies in estimating the further travel requirements of urban area. It develops analytical expressions to predict travel requirements by relating the social and economic status of the area with the demand for transport. The first comprehensive traffic and transportation plan based on such concept has been prepared for Bombay City to solve urban transport problems, and subsequently studies have been extended to other metropolitan cities of Calcutta, Delhi, Bangalore and Madras.

5.3 TRAFFIC OPERATIONS

In order to have safe traffic operations on roads, it is essential to impose adequate traffic regulations and traffic control devices. It is necessary to impress on the public that these regulations and controls are imposed on the public interest to ensure safety in general.

5.3.1 Traffic Regulations

The traffic regulations should cover all aspects of control of vehicles, driver and all other road users. The regulations should be rational. The following are some of the regulations that are enforced from the point of view of safe traffic operations.

Traffic regulations and laws give legal coverage for strict enforcement. The traffic laws implemented by legislative laws are obligatory on all road users. The laws should however be uniform and clear. Traffic regulations and laws cover the following four phases.

(i) *Driver Controls* : These include driving licenses for light and heavy motor vehicles, driver tests and minimum requirements, financial responsibility and civil liability.

(ii) *Vehicle Controls* : The various regulations and controls on vehicles are vehicle registration, requirements of vehicles, equipment and accessories, maximum dimensions and weight and fitness and inspection of vehicles.

(iii) *Flow Regulations* : Regulations of traffic flow have been laid down such as directions, turning and overtaking, etc. In addition control of vehicle operation in traffic stream are made using appropriate regulatory signs like one-way, speed limit prohibitory signs, pedestrian controls, etc.

(iv) *General Controls* : Some other general regulations and provisions are made to report accidents and recording and disposing traffic violation cases.

The *Motor Vehicle Act of 1939* and the several ordinances appending the Act have covered various traffic regulatory measures in India. The various items covered are issue of driving license, registration of vehicles, transfer of ownerships, distinction between private and public vehicles, transport authorities and inter-state commission, limits of speed, weight, parking and halting places, insurance fees, signs, signals and general provisions for punishment of violations and offences.

One-way streets

In congested streets one of the methods to reduce accidents and to ensure smooth flow of traffic is by regulating traffic along *one-way* streets. The traffic is allowed to move only in one specified direction. Such regulations are possible only when there is a net work of roads connecting two bigger roads so that additional distance to be traversed by some vehicles through these one-way streets is not excessive.

The main advantages of one-way streets may be greater capacity, increased average speed, improved pedestrian movement, and reduction in accident. The various types of conflicts at an intersection are

Crossing Conflicts

Merging Conflicts

Diverging Conflicts

On a right angled road intersection with two-way traffic the total number of conflict points are 24. This consists of 16 crossing conflicts which are the major conflict points. The merging and diverging conflicts are considered as minor conflicts, numbering four each in this case, as shown in Fig. 5.21. If one of the roads is declared as one-way, the conflict points decreases to a total of 11, consisting of seven crossing conflicts and four merging conflicts as shown in Fig. 5.22. When both roads are declared as one-way, there are only four crossing conflicts and two merging conflicts, totaling six as shown in Fig. 5.23.

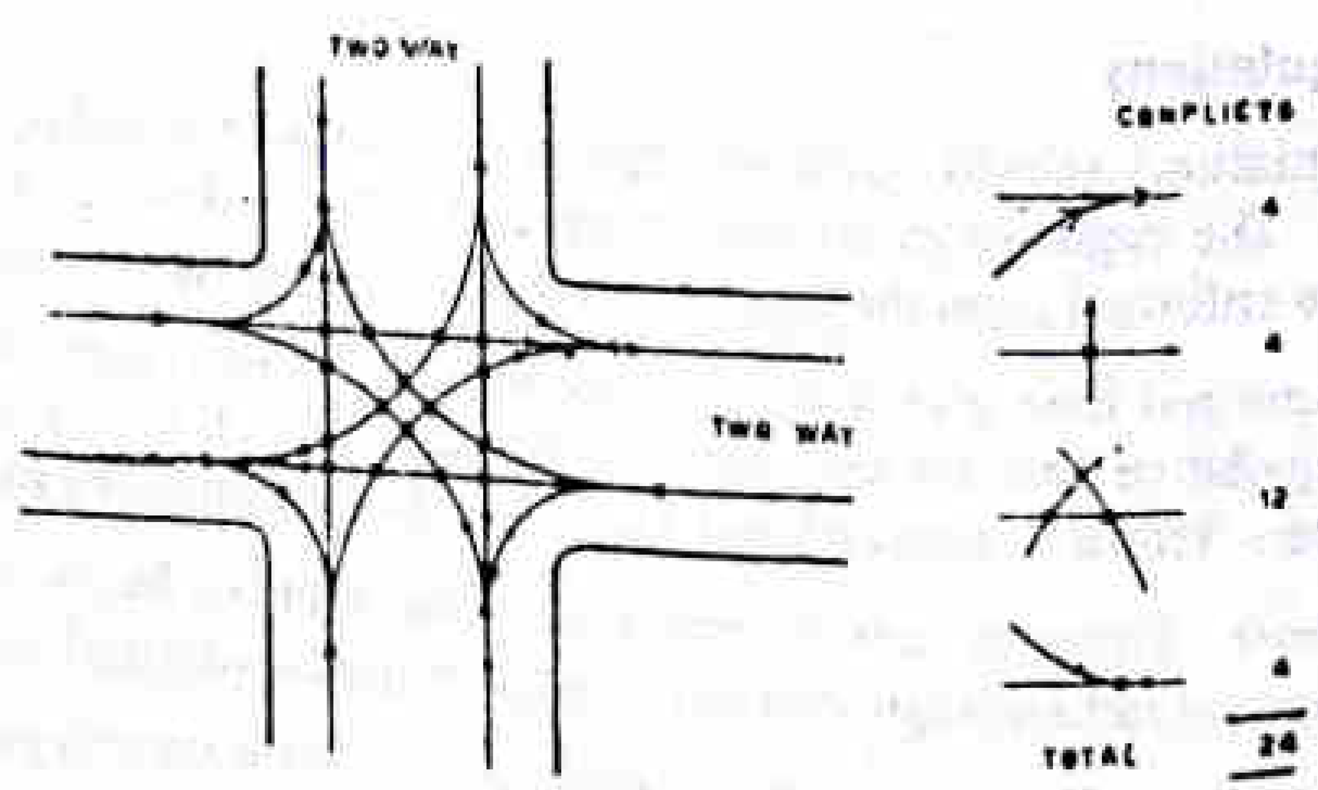


Fig. 5.21 Conflicts with Two-way traffic

Thus the chief advantage of one-way street enforcement is reduction of number of potential points of vehicle conflicts at the uncontrolled intersection as explained above. The potential conflicts and two-way operation and varying number of lanes are given in the following table :

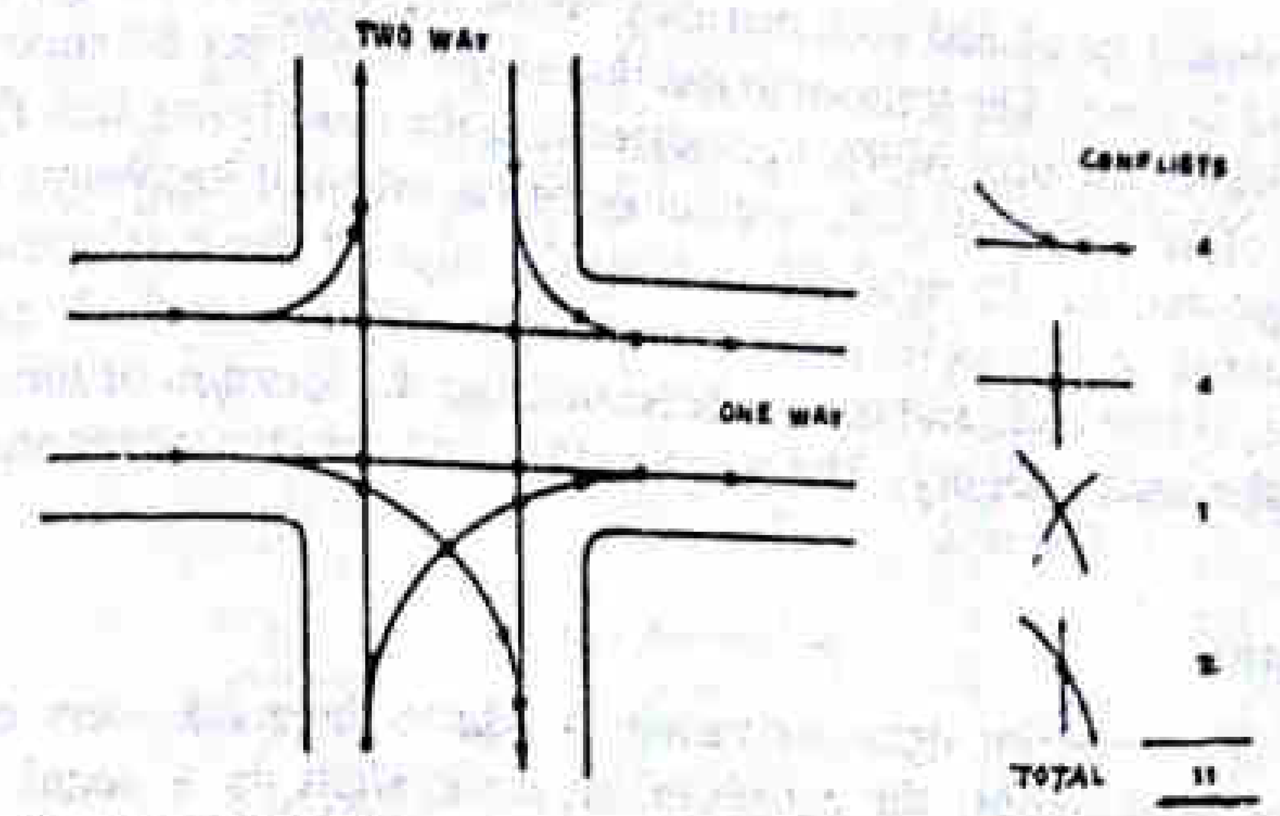


Fig. 5.22 Conflicts with One-way Regulation on One Road

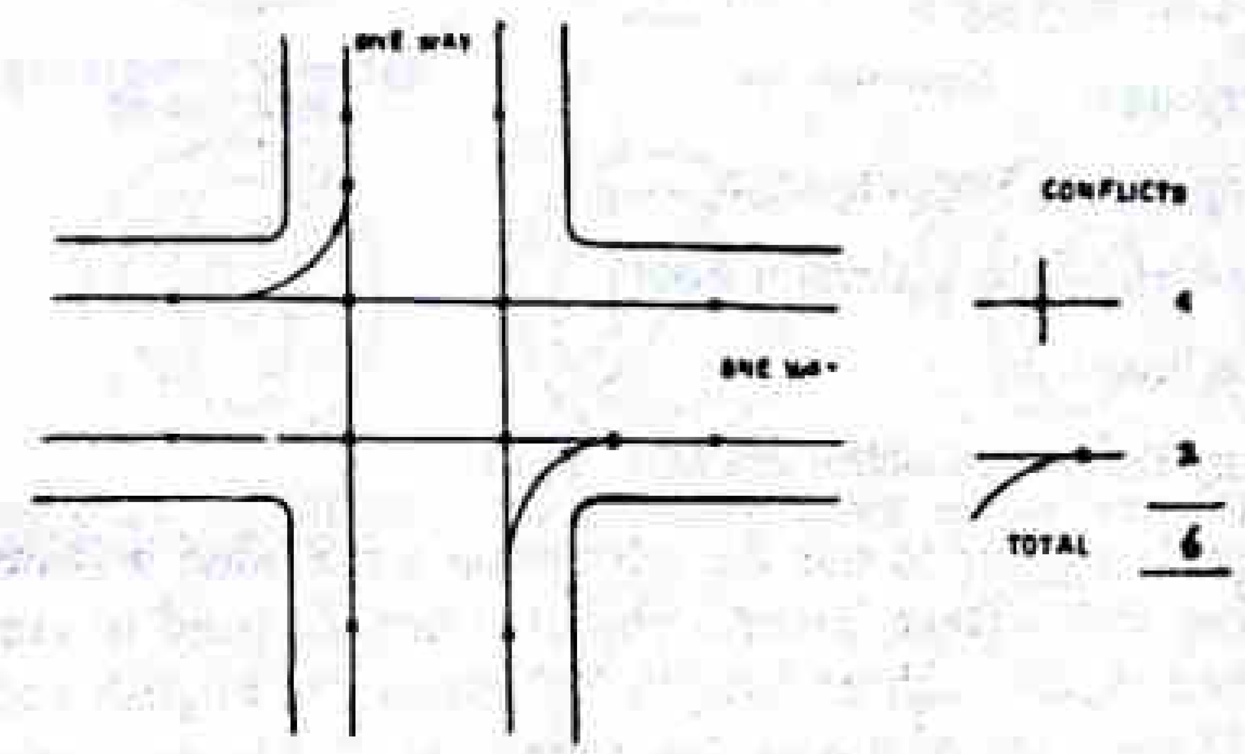


Fig. 5.23 Conflicts with One-way Regulation on Both Roads

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

*3 – lane road when operated as two-way street has one-lane for each direction

5.3.2 Traffic Control Devices

The various aids and devices used to control, regulate and guide traffic may be called traffic control devices. The general requirements of traffic control devices are : attention, meaning, time for response and respect of road users. The most common among these are (a) Signs (b) Signals (c) Markings and (d) Islands. In addition, road lights are useful in guiding traffic during night.

Traffic signs

The traffic signs should be backed by law in order to make them useful and effective. Traffic signs have been divided into three categories according to Indian Motor Vehicles Act. These are (i) Regulatory signs (ii) Warning signs and (iii) Informatory signs.

The signs should be placed such that they could be seen and recognized by the road users easily and in time. The transverse location of the signs may be such that in the case of roads with kerbs, the edge of the sign adjacent to the road is not less than 0.6 m away from the edge of the kerb; on roads without kerbs (as on rural highways with shoulders) the nearest edge may be 2.0 m to 3.0 m from the edge of the carriageway. The signs should be mounted on sign posts painted alternately with 25 cm black and white bands. The size, shape, colour code and the symbols used and the location of the signs should be as specified under each category. The reverse side of all the sign plates should be painted gray.

Regulatory signs

Regulatory or mandatory signs are meant to inform the road users of certain laws, regulations and prohibitions; the violation of these signs is a legal offence. The regulatory signs are classified under the following sub-heads :

(i) Stop and Give-way signs

(ii) Prohibitory signs

(iii) No Parking and No Stopping signs

(iv) Speed Limit and Vehicle Control signs

(v) Restriction Ends sign

(vi) Compulsory Direction Control and other signs

The stop sign is intended to stop the vehicles on a roadway; it is octagonal in shape and red in colour with a white border. This sign may be used in combination with a rectangular definition plate with the word 'STOP' written in English and other languages as necessary. The 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 triangular in shape with the apex downwards and white in colour with a red border; this sign may also be used in combination with a definition plate. These signs are shown in Fig. 5.24.

Prohibitory signs are meant to prohibit certain traffic movements, use of horns or entry of certain vehicle class. These signs are circular in shape and white in colour with a red border. The common prohibitory signs are, Straight Prohibited, No Entry, One-way, Vehicles Prohibited in Both Directions, All Motor Vehicles Prohibited, Truck Prohibited, Bullock Cart and Hand Cart Prohibited, Bullock Cart Prohibited, Tonga Prohibited, Hand Cart Prohibited, Cycle Prohibited, Pedestrian prohibited, Right/Left Turn Prohibited, U-Turn Prohibited, Overtaking Prohibited and Horn Prohibited.

No parking sign is meant to prohibit parking of vehicles at that place, the definition plate may indicate the parking restriction with respect to days, distance, etc. The No Parking sign is circular in shape with a blue back ground, a red border and an oblique red bar at an angle of 45 degrees. No Stopping/Standing sign is meant to prohibit stopping of vehicles at that place; the scope of the prohibition may be indicated on a definition plate. The No Stopping/Standing sign is circular in shape with blue back ground, red border and two oblique red bars at 45 degrees and right angle to each other. The sketches of the Prohibitory Signs, No Parking and No Stopping signs are shown in Fig. 5.24.

Speed Limit signs are meant to restrict the speed of all or certain classes of vehicles on a particular stretch of a road. These signs are circular in shape and have white back ground, red border and black numerals indicating the speed limit. The Vehicle Control

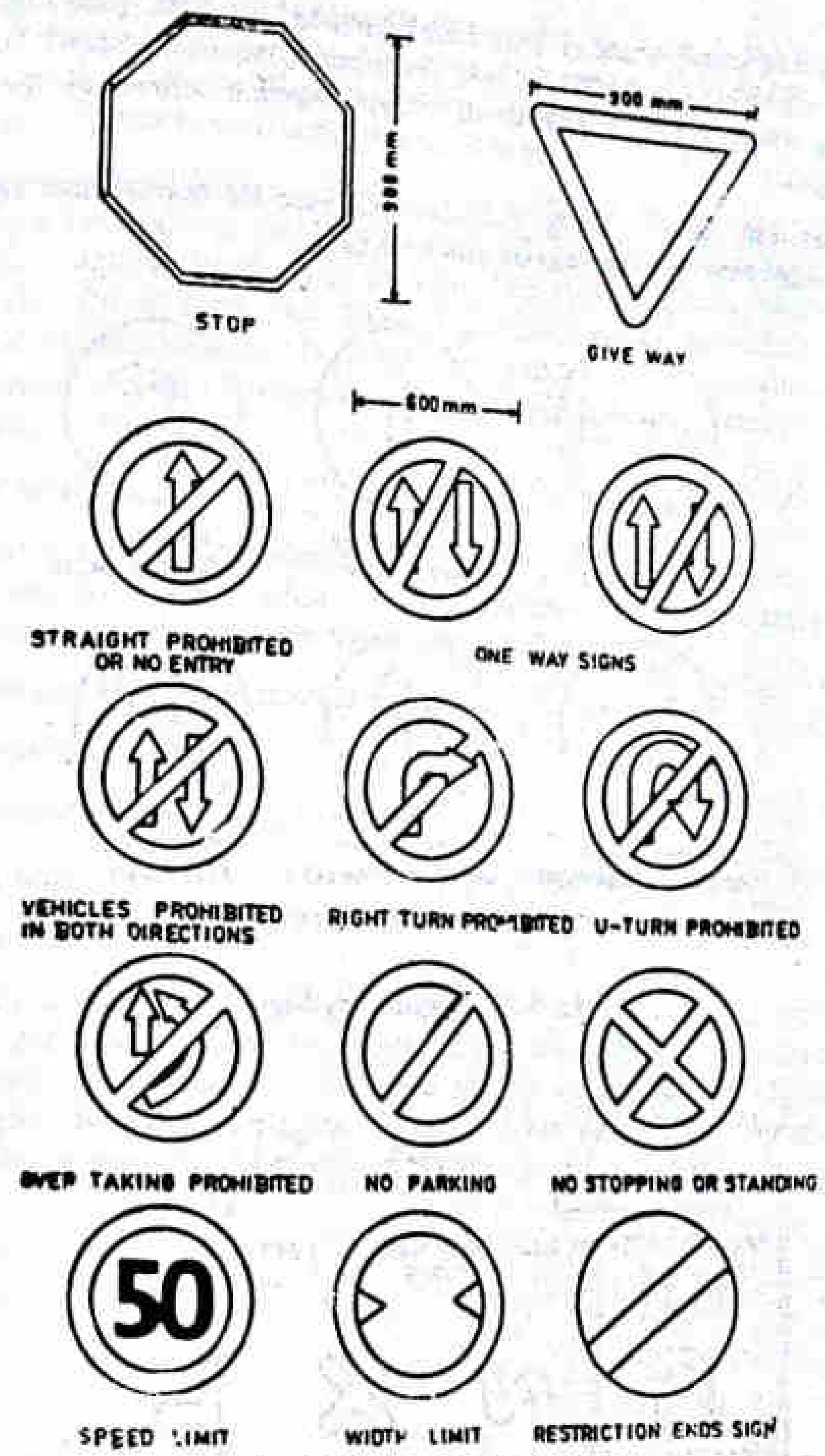


Fig. 5.24 Regulatory Signs

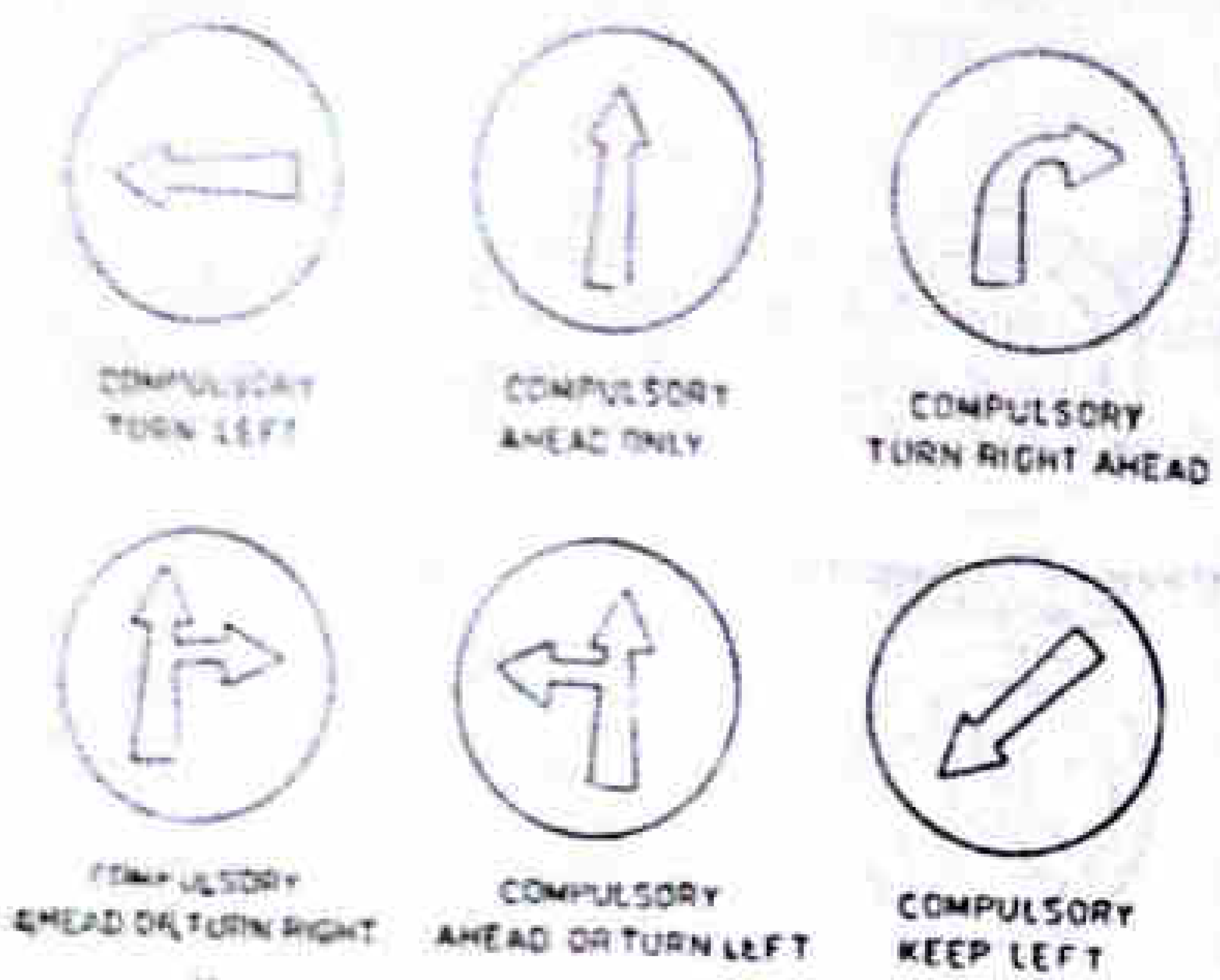
Sings are also similar to Speed Limit signs with black symbols instead of the numerals. The common controls are Width Limit, Height Limit, Length Limit, Load Limit and Axle Load Limit. The definition plate may be used in combination to give more details, symbolically or by words.

Restriction Ends sign indicates the point at which all prohibitions notified by prohibitory signs for moving vehicles case to apply. These signs are also circular with a white back ground and a broad diagonal black band at 45 degrees.

Compulsory Direction Control signs indicate by arrows, the appropriate directions in which the vehicles are obliged to proceed, or the only directions in which they are permitted to proceed. These signs are circular in shape with a blue back ground and white direction arrows. Some of the Compulsory Direction Controls are Compulsory

Turn Left, Ahead Only, Ahead or Turn Left Right and keep Left. (See Fig. 5.25). Other Compulsory signs are Compulsory Cycle Track and Compulsory Sound Horn, [illegible] indicated by white symbols instead of White direction arrows of the Compulsory Direction Signs.

The dimensions shown in Fig. 5.24 and 5.25 are for normal size signs, however smaller size signs may be permitted on minor roads.



(Compusory Direction Control Signs)

Fig. 5.25 Regulatory Signs

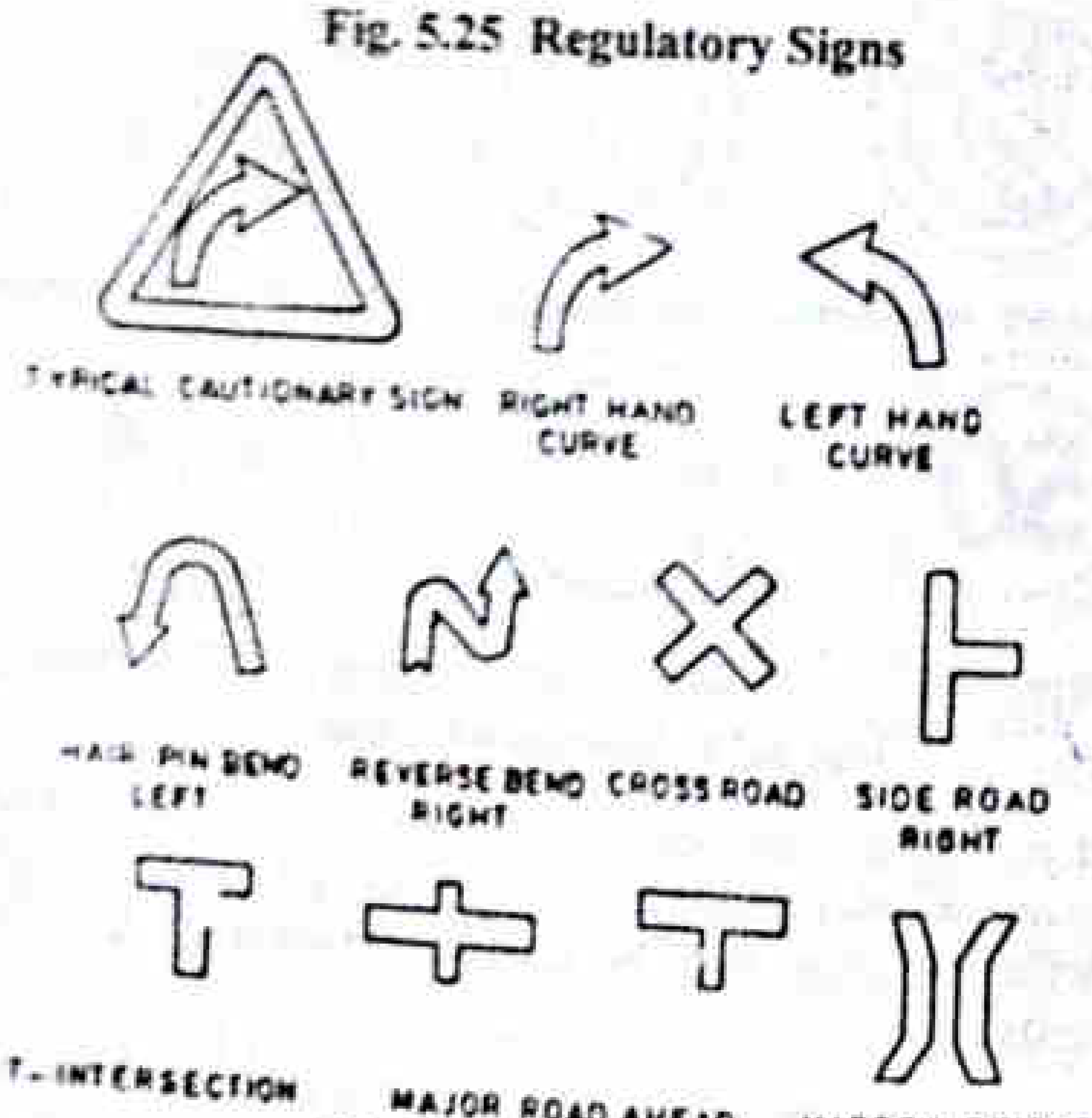


Fig. 5.26 Warning Signs

Warning Signs

Warning or Cautionary signs are used to warn the road users of certain hazardous conditions that exist on or adjacent to the roadway. The Warning signs are in the shape of equilateral triangle with its apex pointing upwards. They have a white back ground,

red border and black symbols. The warning signs are to be located at [illegible] in advance of the hazard warned against. [illegible] respectively on National/State Highways, Major District Roads, Other District Roads and village Roads, on urban roads this distance is 50 metre.

The commonly used Warning signs are, Right Hand/Left Hand Curve, Right/Left Hair Pin Bend, Right/Left Reverse Bend, Steep Ascent/Descent, Narrow Bridge/Road, [illegible] Gap in Median, Slippery Road, Cycle Crossing, Pedestrian Crossing, School Zone, Men at Work, Ferry, Cross Road, Side Road, T-Intersection, Y-Intersection, Major Road Ahead, Round About, Dangerous Dip, Hump or Rough Road, Barrier Ahead, Unguarded Railway Crossing, Guarded Railway Crossing and Falling Rock. Some of these Warning Signs are shown in Fig.5.25.

Informatory sings

These signs are used to guide the road users along routes, inform them of destination and distance and provide with information to make travel easier, safe and pleasant. The information signs are grouped under the following sub-heads:

(i) Direction and Place Identification signs

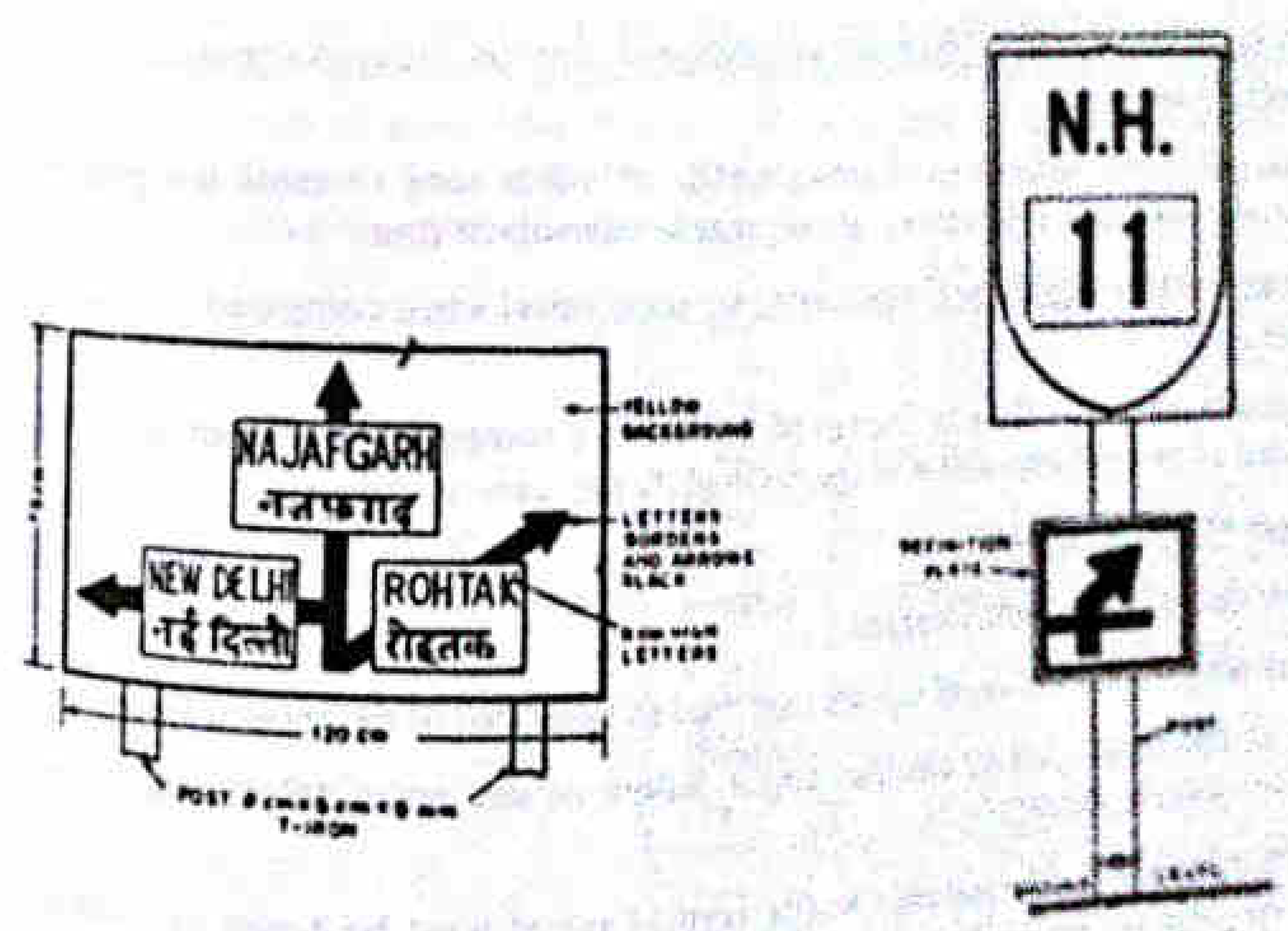
(ii) Facility Information signs

(iii) Other Useful Information signs

(iv) Parking sings

(v) Flood Gauge

The Direction and Place Identification signs are rectangular with white back ground, black border and black arrows and letters. The inscriptions should be in English and other languages as necessary. The signs of this group include Destination signs, Direction signs, Re-assurance signs, Route Marker and Place Identification signs. Figure 5.27 shows some of the Informatory signs.



(a) Road Junction Approach

(b) Route Marker Sign

Fig. 5.27 Informatory Signs

The Facility Information signs are rectangular with blue back ground and white/black letters/symbols. Some of these signs indicate Public Telephone, Petrol Pump, Hospital, First Aid Post, Eating Place and Resting Place. Other useful information signs include No Through Road, No Through Side Road, etc. Parking signs are set up parallel to the road using square sign board with blue black ground and white coloured letter 'P'. Additional definition plate may be used to indicate category of vehicle for which parking space is reserved, direction of parking space etc.

Flood Gauge sign should be installed at all cause ways and submersible bridges or culverts to indicate to the road users the height of the flood above road level.

Traffic signals

At intersection where there are a large number of crossing and right-turn traffic, there is possibility of several accidents as there cannot be orderly movements. The earlier practice has been to control the traffic by means of traffic police by showing stop signs alternately at the cross roads so that one of the traffic streams may be allowed to move while the cross traffic is stopped. Thus the crossing streams of traffic flow are separated by time, seggregation. Traffic signals are control devices which could alternately direct the traffic to stop and proceed at intersections using red and green traffic light signals automatically. The main requirements of traffic signal are to draw attention, provide meaning and time to respond and to have minimum waste of time.

Advantages of traffic signals

Properly designed traffic signals have the following uses :

(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, notably the right angled collisions.

(iii) Pedestrians can cross the roads safely at the signalised intersection.

(iv) The signals allow crossing of the heavy traffic flow with safety.

(v) When the signal system is properly co-ordinated, there is a reasonable speed along the major road traffic.

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

(vii) Automatic traffic signal may workout to be economical when compared to manual control.

(viii) The quality of traffic flow is improved by forming compact platoons of vehicles, provided all the vehicles move at approximately the same speed.

Disadvantages of traffic signals

(i) The rear-end collisions may increase.

(ii) Improper design and location of signals may lead to violations of the control system.

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

The decision to install an automatic traffic control signal must be based on careful analysis of the existing traffic data and on sound engineering judgment. The major emphasis in the criteria for signal control is the volume of traffic entering the intersection and its crossing movements.

The various terms used in traffic signals are briefly explained here. The period of time required for one complete sequence of signal indications is called cycle. A part of the signal cycle allocated to a traffic movement or a combination of traffic movement is called *phase*. Any of the division of the signal cycle during which signal indications do not change is called the *interval*. The engineer has to design the signal with the sequence and duration of individual phases to serve all approaching traffic at a desired *level of service*. The level of service is measured by the vehicle delay, the *queue length* or the number of vehicle backed up and the probability of a vehicle entering the intersection during the first green phase after arrival.

The capacity of a signalised intersection depends on physical factors of the roads such as roadway width, number of lanes geometric design of intersection and also the green and red phases of the traffic signal. In addition, the capacity is affected by operational and control factors such as number of turning movement, number and size of commercial vehicles, pedestrian traffic, peak hour demands, parking regulations, turn control, traffic signal characteristics and abutting land use.

Type of traffic signals

The signals are classified into the following types :

(i) Traffic control signals

(a) Fixed-time signal

(b) Manually operated signal

(c) Traffic actuated (automatic) signal

(ii) Pedestrian signal

(iii) Special traffic signal

The traffic control signals have three coloured light glows facing each direction of traffic flow. The red light is meant for *Stop*, the green light indicates *Go* and the amber or yellow light allows the *clearance time* for the vehicles which enter the intersection area by the end of green time, to clear off. A typical signal head is shown in Fig. 5.28. Additional signals showing green lights for separate movements of turning traffic may also be provided where necessary.

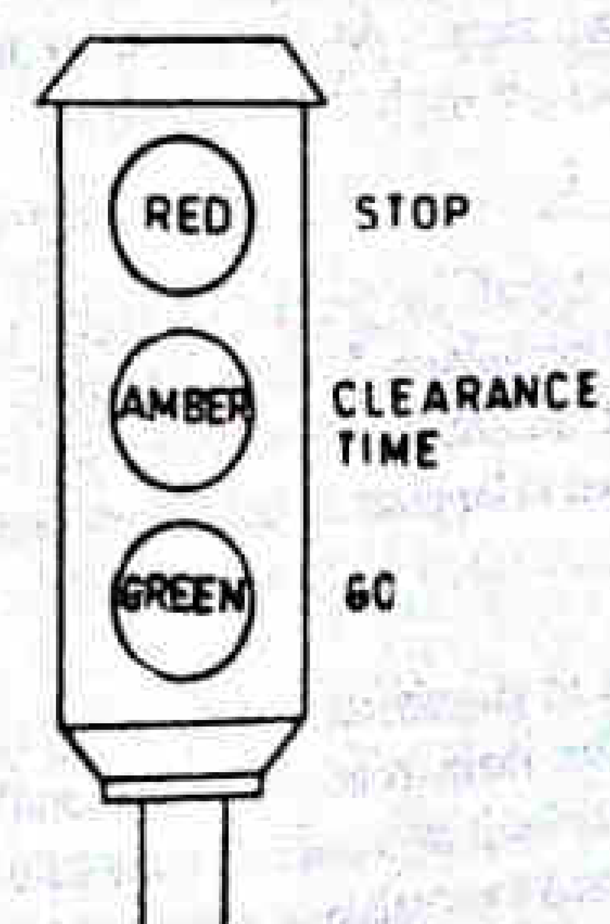


Fig. 5.28 Traffic Signal

Fixed-time signal or pre-timed signals are set to repeat regularly a cycle of red, amber and green lights. The timing of each phase of the cycle is predetermined based on the traffic studies and they are the simplest type of automatic traffic signals which are electrically operated. The main draw back of the signal is that some times the traffic flow on one road may be almost nil and traffic on the cross road may be quite heavy. Yet as the signal operates with fixed timings, the traffic in the heavy stream will have to stop at red phase.

Traffic actuated signals are those in which the timings of the phase and cycle are changed according to traffic demand. In semi-actuated traffic signals the normal green phase of an approach may be extended upto a certain period of time for allowing a few more vehicles approaching closely, to clear off the intersection with the help of detectors installed at the approaches. In fully actuated traffic signals the detectors and a computer assigns the right of way for various traffic movements on the basis of demand and pre-determined programming. But these are very costly to be installed at all intersections.

In some cities in India the traffic police are assigned the duty to watch the traffic demand from suitable observation point during the peak hours on various approaches and to vary the timings of the phases and cycle according to thė actual traffic demand.

When there are series of signals on a city road at each intersection with cross road, the signal system may be operated with only one controller. But it is desirable that a vehicle moving along a main road at normal speed should not have to a stop at every signalised intersection till getting the "go" signal. Hence there should be proper *Co-ordination* of the signal system to provide a *through band.*

Type of traffic signal system

There are four general types of co-ordination of signals for road network, as listed below :

Simultaneous system

Alternate system

Simple progressive system, and

Flexible progressive system

Simultaneous System

In this system all the signals along a given road always show the same indication (green, red etc.) at the same time. As the division of cycle is also the same at all intersections, this system does not work satisfactorily.

Alternate System

In this system, alternate signals or groups of signals show opposite indications in a route at the same time. This system is also operated by a single controller, but by reversing the red and green indicator connections at successive signal systems. This system generally is considered to be more satisfactory than the simultaneous system.

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 signal phases controlling "Go" indications along this road is scheduled to work at the predetermined time schedule. 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.

Flexible Progressive System

In this system it is possible to automatically vary the length of cycle, cycle division and the time schedule at each signalized intersection with the help of a computer. This is the most efficient system of all the four types described above.

Pedestrian signal are meant to give the right of way to pedestrians to cross a road during the 'walk period' when the vehicular traffic shall be stopped by red or stop signal on the traffic signals of the road.

Flashing beacons are meant to warn the traffic. At flashing red signals, the drivers of vehicles shall stop before entering the nearest cross walk at an intersection or at a stop line, when marked Flashing yellow signals are caution signals meant to signify that drivers may proceed with caution.

Warrants for traffic control signal installation

Traffic control signals should not be installed unless one or more of the following signal warrants are met. The necessary data should be collected by means of traffic-engineering studies :

(i) Minimum vehicular volume warrant specifies that the average traffic volume for eight hours on both approaches should be atleast 650 motor vehicles per hour on major streets with single lane and 800 vehicles on the streets with two or more lanes. Further the number of motor vehicles approaching the intersection on minor street (on one direction only) is atleast 200 vehicles per hour on single lane street and 250 vehicles per hour when there are two or more lanes. However when the average approach speed or the 85th percentile speed on major street exceeds 60 kmph or when the intersection lies within built-up area, the vehicular volume warrant may be decreased to 70 percent of the above requirement.

(ii) Interruption of continuous traffic flow on the major street with 1000 to 1200 vehicles per hour that there is undue delay or hazard to traffic on minor road with a traffic of 100 to 150 vehicles per hour in one direction only during any eight hours of an average day.

(iii) Minimum pedestrian volume warrant of 150 or more pedestrians per hour cross a major street with over 600 vehicles per hour on both approaches, (1000 vehicles per hour in the case of main street with raised median). However when the average approach speed or the 85th percentile speed exceed 60 kmph, 70 percent of the above requirements may be adopted.

(iv) Accident experience warrant showing that other measures have failed to decrease the accident frequency or when five or more accidents (involving over Rs. 2000 due to injury and damage) have occurred within 12 months period. However signal installation should not seriously disrupt the traffic flow.

(v) Combination of warrants when no single warrant is satisfied but indicating two or more warrants of (i), (ii) or (iii) above are satisfied to the extent of 80 percent or more of the stated volume.

Design of isolated fixed time signal

In the design of a signalised intersection, the objective should be to provide sufficient capacity for the volume of traffic approaching the intersection. The design should aim at minimising total delay, building short queues, and providing a high probability of passing through the intersection on the first given period for most users. Signal timing should be in accordance with traffic flow on intersection. The cycle lengths are normally 40 to 60 seconds for two phase signal. Longer cycle lengths are in use for complex traffic flow and for more than two phases.

General Principles of Signal Design

(i) Stop time or red phase R1 of a signal is the sum of go and clearance intervals or green and amber phases for the cross flow i.e., $G_2 + A_2$ at a two phase signal. During this interval, the pedestrian crossing time may also be incorporated for the road, if turning movements are not permitted.

(ii) Towards the end of red phase, there may be a short duration when the amber lights are put-on along with red light signal in order to indicate 'get set' to go. This phase is the last part of red phase itself and may be called 'red-amber' or 'initial amber'. The vehicles are not supposed to cross the stop line during the red amber period.

(iii) Clearance time or clearance amber phase is provided just after the green phase before the red phase, to fulfil two requirements :

(a) Stopping time for approaching vehicle to stop at stopline after the signal changes from green to amber and not to cross the line by the time the signal changes to red phase.

(b) Clearance time for the vehicle which is approaching the stop line at legal speed while the signal changes from green to amber, allowing sufficient time for the vehicle to cross the intersection area as it may not be possible for the vehicle to stop before the stop line at that stage. Usually 2.0 to 4.0 seconds would be suitable for the amber phase.

(iv) Go time or green time is decided based on the approach volume during peak hour and to enable the queued vehicles to clear off in most of the cycles.

Two approximately design procedures (viz., trial cycle method and approximate method) and one rational approach (Webster's method) for the design of traffic signal cycles are given below for fixed time traffic signals at cross roads. In addition the signal design method as per the guidelines of the IRC is also given. For the purpose of simplicity, two phase traffic signals with no turning movements are illustrated here. The methods may be suitably extended for multiphase operation.

(1) Trial cycle method

The 15 minute-traffic counts n_1 and n_2 on road 1 and 2 are noted during the design peak hour flow. Some suitable trial cycle C_1 second is assumed and the number of the assumed cycles in the 15 minutes or 15×60 seconds period is found to be $(15 \times 60)/C_1$ i.e. $(900/C_1)$. Assuming an average time headway 2.5 seconds, the green periods G_1 and G_2 of roads 1 and 2 are calculated to clear the traffic during the trial cycle.

$$G_1 = \frac{2.5\,n_1\,C_1}{900} \text{ and } G_2 = \frac{2.5\,n_2\,C_2}{900}$$

The amber periods A_1 and A_2 are either calculated or assumed suitably (3 to 4 seconds) and the length C_1' is calculated, equal to $(G_1 + G_2 + A_1 + A_2)$ seconds. If the calculated cycle length C_1' works out to be approximately equal to the assumed cycle length C_1, the cycle length is accepted as the design cycle. Otherwise the trials are repeated till the trial cycle length works out approximately equal to the calculated value.

Example 5.13

The 15 minute-traffic counts on cross roads 1 and 2 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 loads 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 (i)

Assume a trial cycle $C_1 = 50$ secs.

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

Green time for road 1, allowing an average time headway of 2.5 secs. per vehicle

$$G_1 = \frac{178 \times 2.5}{18} = 24.7 \text{ secs.}$$

$$\text{Green time for road 2, } G_2 = \frac{142 \times 2.5}{18} = 19.7 \text{ secs.}$$

Amber times A_1 and A_2 are 3 and 2 secs. (given)

$$\text{Total cycle length} = 24.7 + 19.7 + 3.0 + 2.0 = 49.4 \text{ secs.}$$

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

Trial (ii)

Assume trial cycle $C_2 = 40$ secs.

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

$$\text{Green time for road 1, } G_1 = \frac{178 \times 2.5}{22.5} = 19.8 \text{ secs.}$$

$$\text{Green time for road 2, } G_2 = \frac{142 \times 2.5}{22.5} = 15.8 \text{ secs.}$$

$$\text{Total cycle length} = 19.8 + 15.8 + 3 + 2 = 40.6 \text{ secs.}$$

Trial (iii)

Assume trial cycle $C_3 = 45$ secs.

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

$$\text{Green time for road 1, } G_1 = \frac{178 \times 2.5}{20} = 22.25 \text{ secs.}$$

$$\text{Green time for road 2, } G_2 = \frac{142 \times 2.5}{20} = 17.75 \text{ secs.}$$

$$\text{Total cycle length} = 22.25 + 17.75 + 3 + 2 = 45.0 \text{ secs.}$$

Therefore the trial cycle of 45 secs. may be adopted with the following signal phases :

$G_1 = 22.25$, $G_2 = 17.75$, $A_1 = 3.0$, $A_2 = 2.0$ and Cycle length = 45.0 secs.

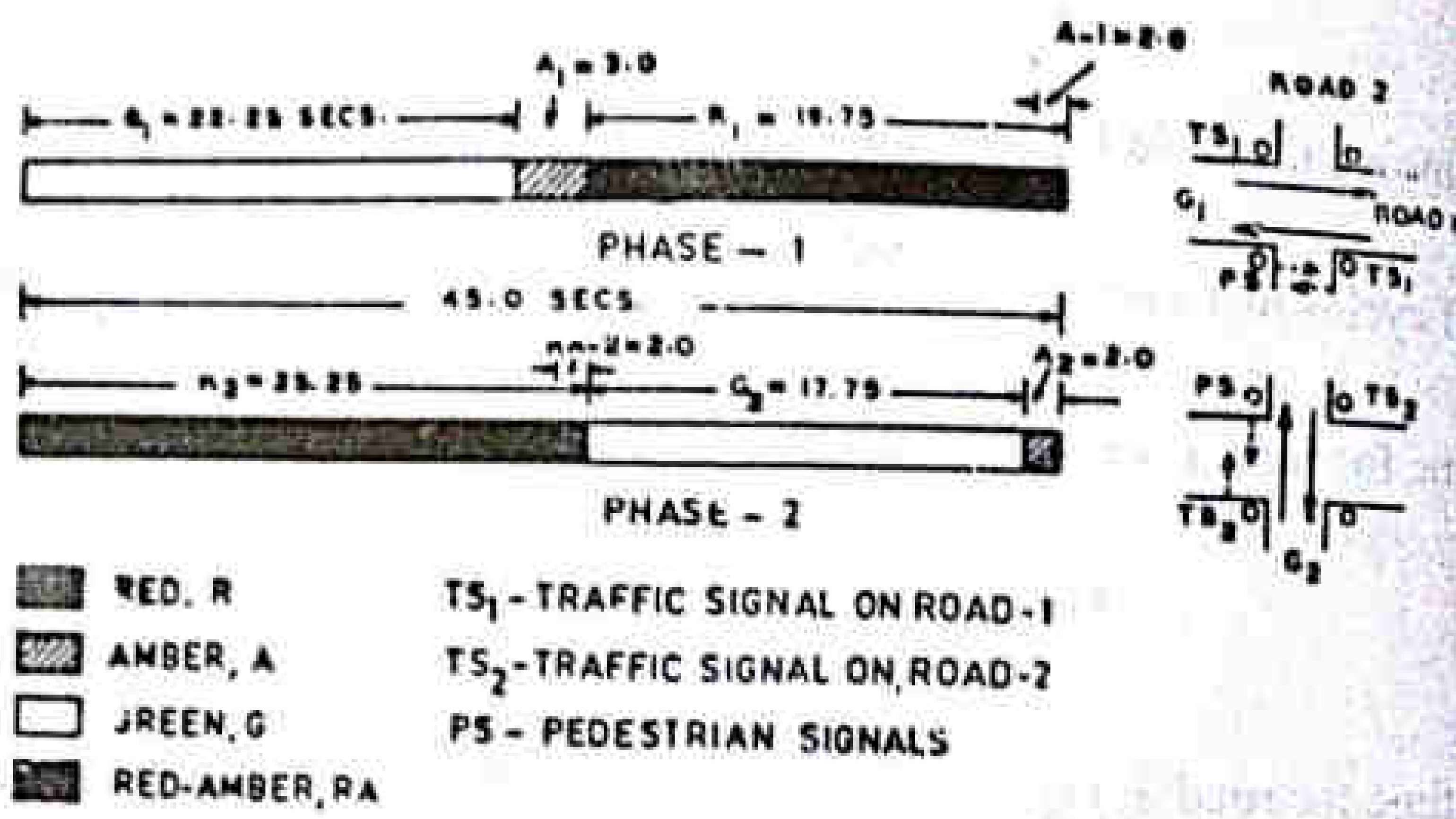


Fig. 5.29 Phase Diagram and Details of Signal Setting (Example 5.13)

(2) Simple design of pedestrian and traffic signals by approximate method

The following design procedure is suggested for the simple design of a two phase signal unit at cross roads, along with pedestrian signals :

(i) Based on approach speeds of the vehicles, the suitable, clearance interval between green and red period i.e., clearance amber periods are selected. The amber periods may be taken as 2, 3 and 4 seconds for low, medium and fast approach speeds.

(ii) Based on pedestrian walking speed of 1.2 m per second, the clearance for pedestrian time is also calculated.

(iii) Minimum red time of traffic signal is taken as pedestrian clearance time for crossing plus initial interval for pedestrians to start crossing. This red time is equal to the minimum green time plus amber time for the cross road.

(iv) The minimum green time is calculated based on pedestrian criterion, equal to pedestrian clearance time for cross road plus an initial interval when pedestrians may start to cross minus amber period. This is equal to red time for cross road minus amber period for the cross road.

(a) with pedestrian signal the initial interval is the WALK period; this should not be less than seven seconds.

(b) where no pedestrian signal is used, a minimum period of five seconds is used as initial interval.

(v) The actual green time needed is then increased based on the ratio of approach volume for the heaviest traffic volume per hour per lane. The cycle length so obtained is adjusted for the next higher 5-second interval. The extra time is then distributed to green timings in proportion to the approaching volumes of traffic.

(vi) The values so obtained are calculated on percentage basis if the controller settings are in per cent of cycle.

(vii) The timings so obtained are installed in the controller and the operations are then observed at the site during peak traffic hours. Corrections or modifications are carried out if needed.

The design of a simple two-phase signal is illustrated by an example below.

Example 5.14

An isolated signal with pedestrians indication is to be installed on a right angled intersection with road A 18 m wide and road B, 12 m wide. The heaviest volume per hour for each lane of road A and road B are 275 and 225, respectively. The approach speeds are 55 and 40 kmph, for A and road B respectively. Design the timings of traffic and pedestrian signals.

Solution

The layout of traffic and pedestrian signals is shown in Fig. 5.30.

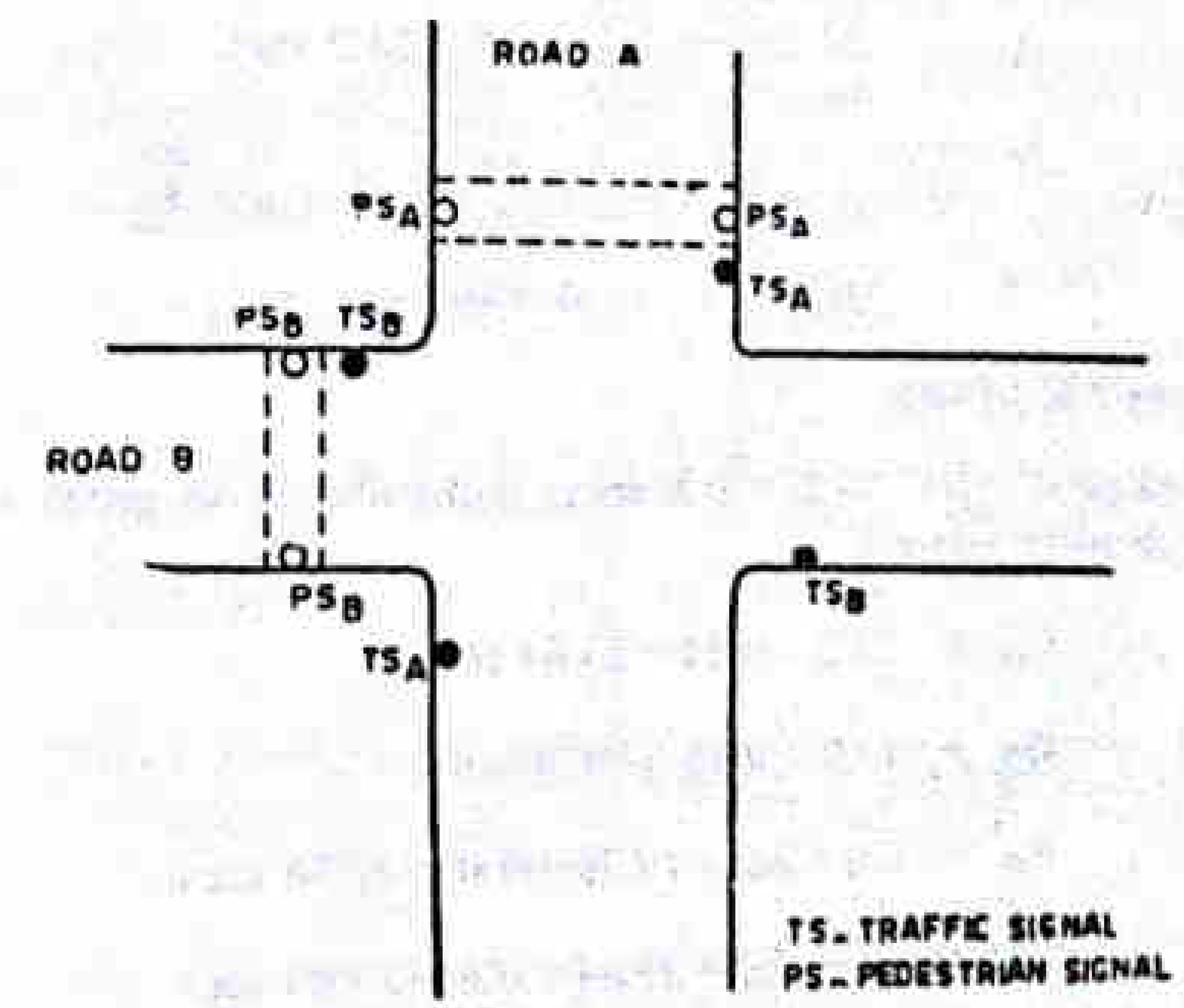


Fig. 5.30 Placement of Signals (Example 5.14)

Design of traffic signals

(i) Based on the approach speed; amber periods :

For road A with 55 kmph, amber period, $A_A = 4$ secs.

For road B with 40 kmph, amber period, $A_B = 3$ secs.

(ii) Based on the pedestrian walking speed of 1.2 m/sec, the pedestrian clearance time is calculated.

$$\text{Road A} = \frac{18}{1.2} = 15 \text{ seconds;}$$

$$\text{Road B} = \frac{12}{1.2} = 10 \text{ seconds}$$

(iii) Adding 7 secs. for initial walk-period, minimum red time for road A is (15 + 7) secs. and that for road B is (10 + 7) secs.

(iv) Minimum green times based on pedestrian criterion :

$$\text{Road B} = 15 + 7 - 3 = 19 \text{ secs.}$$

$$\text{Road A} = 10 + 7 - 4 = 13 \text{ secs.}$$

(v) Based on approach volume, the green time calculated is increased for Road A with higher traffic volume.

Use relation $\dfrac{G_A}{G_B} = \dfrac{n_A}{n_B}$

G_A and G_B are green times and n_A and n_B are approach volume per lane

G_B is taken as 19 seconds as in (iv) above.

Green time for Road A,

$$G_A = \frac{n_A}{n_B} G_B = \frac{275}{225} \times 19 = 23.2 \text{ secs.}$$

(vi) Total cycle length

$$= G_A + A_A + R_A = G_A + A_A + G_B + A_B$$

$$= 23.2 + 4 + 19 + 3 = 49.2 \text{ secs.}$$

Hence adopt cycle length of 50 secs.

The additional period of 50 − 49.2 = 0.8 secs. is distributed to green timings in proportion to approach traffic volume.

$$G_A = 23.2 + 0.44 = 23.64 \text{ secs.}$$

$$G_B = 19.0 + 0.36 = 19.36 \text{ secs.}$$

$$R_A = G_B + A_B = 19.36 + 3.0 = 22.36 \text{ secs.}$$

$$R_B = G_A + A_A = 23.64 + 4.0 = 27.64 \text{ secs.}$$

Design of pedestrian signal

Do not Walk (DW) period of pedestrian signal at road A (PS_A) is red period of traffic signal at B.

For PS_A, $\quad DW_A = R_B = 27.64$ secs.

For PS_B, $\quad DW_B = R_A = 22.36$ secs.

Pedestrian clearance intervals (CI) are of 15 and 10 secs. respectively, for roads A and B for crossing from (ii) above. The walk time (W) is calculated from total cycle length.

For PS_A, $\quad W_A = 50 - (27.64 + 15) = 7.36$ secs.

For PS_B, $\quad W_B = 50 - (22.36 + 10) = 17.64$ secs.

Details of design timings are tabulated in Table 5.9. Alternatively a phase diagram may be drawn, as shown in Fig. 5.31.

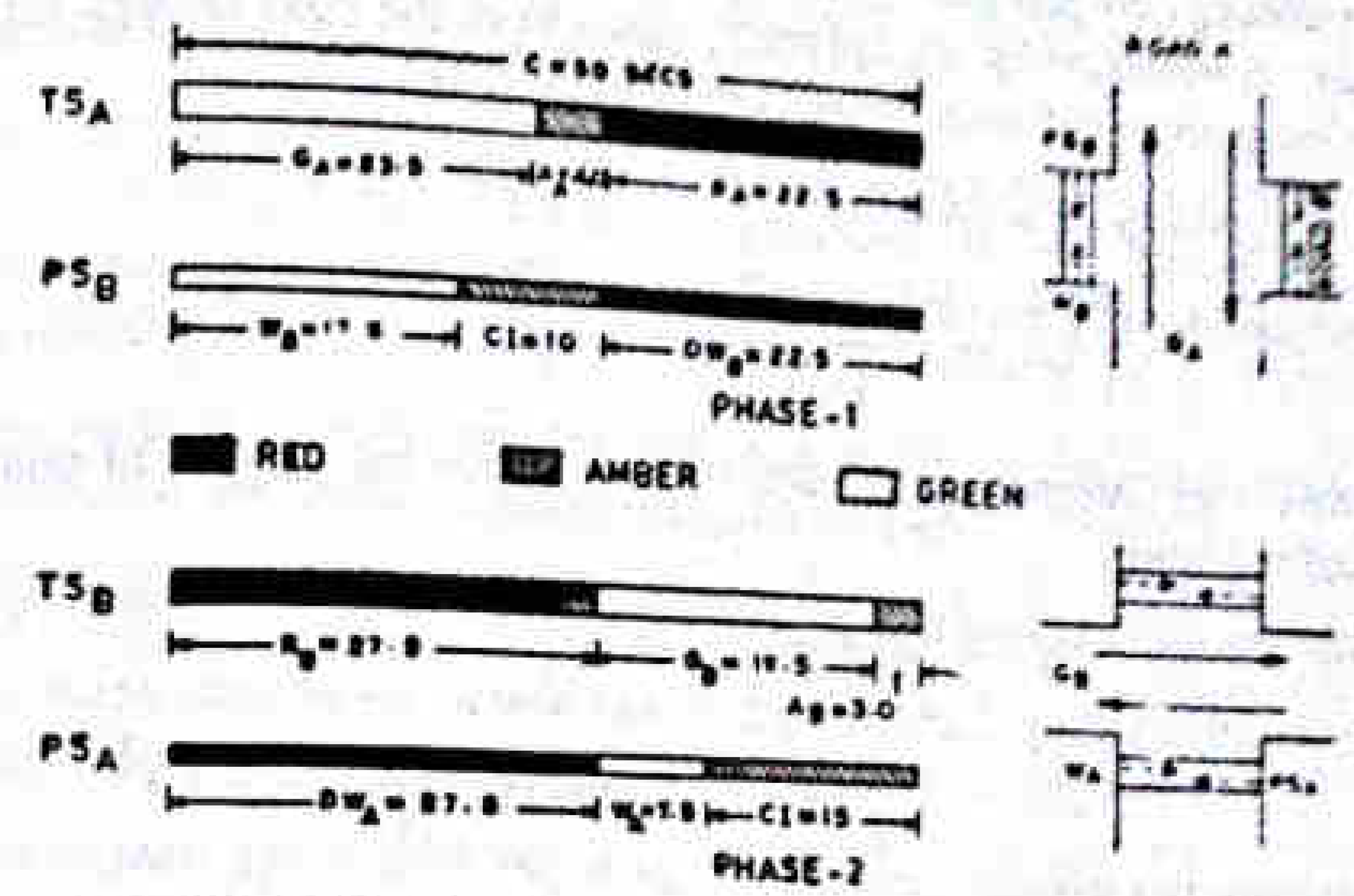


Fig. 5.31 Traffic and Pedestrian Signal Settings

Table 5.9 Details of Design Timings (Ex. 5.13)

Interval	Road A Vehicle TS_A	Road A Pedestrian PS_B	Road B Vehicle TS_B	Road B Pedestrian PS_A	Time Interval (%)	Actual time, interval seconds: Pedestrian	Actual time, interval seconds: Traffic
1	G23.64	W 17.64	R 27.64	DW 27.64	35	W_B 17.5	$G_A = 23.5$ } $R_B = 27.5$
2		CI 10.00			12	$DW_A = 27.5$; W_B 6.0	
3	A 4.00				8	CI_B } 4.0 A_A	
4				W 7.36	15	W_A 17.5	
5	R 22.36	DW 22.36	G19.36	CI 15.00	24	$DW_B = 22.5$; W_A 12.0	$G_B = 19.5$ } $R_A = 22.5$
6			A 3.00		6	CI_A } 3.0 A_B	
Total	50.0		50.0	50.0	100%	50.0 secs	50.0

Traffic signal
R-red (stop)
A-amber (clearance)
G-green (go)

Pedestrian signal
W-walk
CI-clearance for pedestrians (may be indicated by flickering of walk signal W)
DW-do not walk

Note : Amber period of a short interval however be provided in the traffic signal in between red and green periods for getting ready to cross. This period, is a part of the red period itself

(3) Webster's method

In this method, the optimum signal cycle C_o corresponding to least total delay to the vehicles at the signalized intersection has been worked out. This is a rational approach. The field work consists of finding (i) the saturation flow S per unit time on each approach of the water section and (ii) the normal flow q on each approach during the design hour. Based on the higher value of normal flow, the ratio $y_1 = q_1/S_1$ and $y_2 = q_2/S_2$ are determined on the approach roads 1 and 2. In the case of mixed traffic, it is necessary to convert all the normal flow and saturation flow values in terms of suitable PCU values which should be determined separately.

The saturation flow is to be obtained from careful field studies by noting the number of vehicles in the stream of compact flow during the green phases, and the corresponding time intervals precisely. In the absence of data the approximate value of saturation flow is estimated assuming 160 pcu per 0.3 metre width of the approach. The normal flow of the traffic is also determined on the approach roads from the field studies for the design period (during the peak or off-peak hours as the case may be).

The optimum signal cycle is given by :

$$C_o = \frac{1.5L+5}{1-Y} \tag{5.24}$$

where L = total lost time per cycle, secs. = 2n + R (n is the number of phase and R is all red-time)

$$Y = y_1 + y_2$$

Then,

$$G_1 = \frac{y_1}{Y}(C_o - L) \text{ and } G_2 = \frac{y_2}{Y}(C_o - L) \tag{5.25}$$

Similar procedure is followed when there are more number of signal phases.

Example 5.15

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

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

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

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

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

$$C_o = \frac{1.5L+5}{1-Y} = \frac{1.5\times16+5}{1-0.57}$$

$$= \frac{29}{0.43} = 67.4 \text{ say, } 67.5 \text{ secs.}$$

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

$$G_b = \frac{y_b}{Y}(C_o - L) = \frac{0.25}{0.57}(67.5 - 16) = 22.5 \text{ secs.}$$

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.

Note : Sketch a phase diagram as shown in Fig. 5.29 or 5.31 or prepare a table to show the signal phases as given in Table 5.9

(4) Design Method as per IRC Guideline

(i) The pedestrian green time required for the major and minor roads are calculated based on walking speed of 1.2 m/sec. and initial walking time of 7.0 secs. These are the minimum green time required for the vehicular traffic on the minor and major roads respectively.

(ii) The green time required for the vehicular traffic on the major road is increased in proportion to the traffic on the two approach roads.

(iii) The cycle time is calculated after allowing amber time of 2.0 secs. each.

Note : The steps mentioned above are similar to the Approximate Method explained under Method (2) and Example 5.14.

(iv) The minimum green time required for clearing vehicles arriving during a cycle is determined for each lane of the approach road assuming that the first vehicle will take 6.0 secs. And the subsequent vehicles (PCU) of the queue will be cleared at a rate of 2.0 secs. The minimum green time required for the vehicular traffic on any of the approaches is limited to 16 secs.

(v) The optimum signal cycle time is calculated using Webster's formula (explained in method 3). The saturation flow values may be assumed as 1850, 1890, 1950, 2250, 2550 and 2990 PCU per hour for the approach roadway widths (kerb to median or centre line) of 3.0, 3.5, 4.0, 4.5, 5.0 and 5.5 m; for widths above 5.5 m, the saturation flow may be assumed as 525 PCU per hour per metre width. The lost time is calculated from the amber time, inter-green time and the initial delay of 4.0 secs. for the first vehicle, on each leg.

(vi) The signal cycle time and the phases may be revised keeping in view the green time required for clearing the vehicles and the optimum cycle length determined it steps (iv) and (v) above.

The design method is illustrated in Example 5.16.

Example 5.16

At a right angled intersection of two roads, Road 1 has four lanes with a total width of 12.0 m and Road 2 has two lanes with a total width of 6.6 m. The volume of traffic approaching the intersection during design hour are 900 and 743 PCU/hour on the two approaches of Road 1 and 278 and 180 PCU/hour on the two approaches of Road 2. Design the signal timings as per IRC guidelines.

Solution

Design traffic on Road 1 = higher of the two approach volume per lane = 900/2 = 450 PCU/hr.

Design traffic on road 2 = 278 PCU/hr

(i) Pedestrian green time for Road 1 $= \frac{12.0}{1.2} + 7.0 = 17$ secs.

Pedestrian green time for Road 2 $= \frac{6.6}{1.2} + 7.0 = 12.5$ secs.

Green time for vehicles on Road 2, G_2 = 17.0 secs.

(ii) Green time for Road 1, $G_1 = 17 \times \frac{450}{278} = 27.5$ secs.

(iii) Adding 2.0 secs. each towards clearance amber and 2.0 secs. inter-green period for each phase, total cycle time required $= (2 + 17 + 2) + (2 + 27.5 + 2) = 52.2$ secs.

Signal cycle time may be conveniently set in multiples of five secs. and so the cycle time = 55 secs.

The extra 2.5 secs. per cycle may be apportioned to the green times of Roads 1 and 2 as 1.5 and 1.0 secs. and so $G_1 = 27.5 + 1.5 = 29.0$ secs. and $G_2 = 17.0 + 1.0 = 18.0$ secs.

(iv) Vehicle arrivals per lane cycle on Road 1

$$450/55 = 8.2 \text{ PCU}$$

Minimum green time for clearing vehicles on Road 1

$$= 6 + (8.2 - 1.0)\,2 = 20.4 \text{ secs.}$$

Vehicle arrivals per cycle on Road 2

$$= 278/55 = 5.1 \text{ PCU}$$

Minimum green time for clearing vehicles on Road 2

$$= 6 + (5.1 - 1.0)\,2 = 14.2 \text{ secs.}$$

As the green time provided for the two roads by pedestrian crossing criteria in (iii) above are higher than these values, the above design values are alright.

(v) Lost time per cycle = (amber time + inter-green time + time lost for initial delay of first vehicle) for two phases = $(2 + 2 + 4) \times 2 = 16$ sec.

Saturation flow for Road 1 $= 525 \times 6 = 3150$ PCU/hr

Saturation flow for Road 2 $= 1850 + \frac{40 \times 3}{5} = 1874$ PCU/hr

$$y_1 = \frac{900}{3150} = 0.286 \text{ and } y_2 = \frac{278}{1874} = 0.148$$

$$Y = 0.286 + 0.148 = 0.434$$

Optimum cycle time

$$C_o = \frac{1.5L + 5}{1 - Y} = \frac{1.5 \times 16 + 5}{1 - 0.434} = 51.2 \text{ secs.}$$

Therefore the cycle time of 55 secs. designed earlier is acceptable. The details of the signal timings are given below. These may either be shown in the form of phase diagram as in Fig. 5.29 and 5.31 or in a tabular form as in Table 5.9.

Road	Green	Amber	Red	Cycle
Road 1	29	2	(22 + 2)	55
Road 2	18	2	(33 + 2)	55

Road marking

Road or traffic markings are made of lines, patterns, words, symbols or reflectors on the pavement, kerb, sides of islands or on the fixed objects within or near the roadway. Traffic markings may be called special signs intended to control, warn, guide or regulate the traffic. The markings are made using paints in contrast with colour and brightness of the pavement or other back ground. Light reflecting paints are also commonly used for traffic marking. In order to ensure that the markings are seen by the road users, the longitudinal lines should be atleast 10 cm thick and the transverse lines should be made in such a way that they are visible at sufficient distance in advance to give road users adequate time to respond.

The various types of markings may be classified as,

(a) Pavement markings

(b) Kerb markings

(c) Object markings and

(d) Reflector unit markings

Pavement Markings

Pavement or carriageway markings may generally be of white paint. Yellow colour markings are used to indicate parking restrictions and for the continuous centre line and barrier line markings. Longitudinal solid lines are used as guiding or regulating lines and are not meant to be crossed by the driver. Transverse solid lines indicate the position of stop lines for vehicular traffic.

Some of the common types of pavement markings are given below :

(a) *Centre Lines* : These are meant to separate the opposing streams of traffic on undivided two-way roads. On rural highway with two or three lanes, single broken lines of width 0.1 m and length 4.5 segments and 7.5 m gaps may be painted on straight stretches of NH and SH, these may be decreased to 3.0 and 6.0 m at horizontal curves and approaches to intersection. On other roads at straights the segments are 3.0 m in length and gaps 6.0 m (which are reduced to 3.0 m at curves and approaches to intersection). On four or six lane undivided roads two solid continuous parallel lines of 0.1 m width with 0.05 to 0.10 m space in between are painted.

On urban roads with less than four traffic lanes the centre line consists of white broken lines of width 0.10 to 0.15 m, length of segment 3.0 m and length of gaps 4.5 m to be reduced to 3.0 m at curves and approaches to intersections. On undivided roads with atleast two traffic lanes for each direction of traffic flow, the centre line marking shall consist of two solid continuous lines.

(b) *Lane Line* : Lines are drawn to designate traffic lanes. These are used to guide the traffic and to properly utilize the carriageway.

(c) *No Passing Zone Markings* : These are marked to indicate that overtaking is not permitted.

(d) *Turn Markings* : These are useful near intersection to designate proper lateral placement of vehicles before turning to the different directions.

(e) *Stop Lines* : These are meant for vehicles to stop near the pedestrian crossing, signalized intersection etc. where the vehicles have to stop and proceed.

(f) *Cross Walk Lines* : The particular places where pedestrian are to cross the pavement are properly marked by the pavement markings. The width of pedestrian crossing may be between 2.0 and 4.0 m depending on local requirements.

(g) *Approach to Obstructions*: These may be indicated by appropriate pavement markings.

(h) *Parking Space Limits*: For proper utilization of parking facility, markings are made.

(i) Border or edge lines indicate carriageway edges of rural roads which have no kerb stones along the edges.

(j) Route direction arrows are marked by one or more arrows to guide effectively the traffic into correct lanes.

(k) Parking space limits on urban roads are marked to promote efficient use of parking spaces in a systematic manner.

(*l*) Bus Stops : The length of kerb which is reserved for buses to stop are marked by continuous yellow line on the kerb indicating 'parking prohibited'. The pavement space meant for bus stop is also marked by the word 'BUS'.

Kerb Markings

These may indicate certain regulations like parking regulations. Also the markings on the kerb and edges of islands with alternate black and white line increase the visibility from a long distance.

Object Markings

Physical obstruction on or near the roadway are hazardous and hence should be properly marked. Typical obstructions are supports for bridge, signs and signals, level crossing gates, traffic islands, narrow bridges, culver head walls etc.

Reflector Unit Markings

Reflector markers are used as hazard markers and guide markers for safe driving during night. Hazard markers reflecting yellow light should be visible from a long distance of about 150 m.

Road Delineators

Road delineators are devices or treatment to outline the roadway or a portion there-of to provide visual assistance to drivers about the alignment of a road ahead, especially at night. Three types of delineators that may be used are Roadway Indicators, Hazard Markers and Object Markers.

Roadway indicators are in the form of guide posts, 0.8 to 1.0 m high and painted by black and white strips with or without reflectors and are intended to delineate the edges of the roadway so as to guide the drivers about the alignment ahead. Hazard markers are approximately 1.2 m high plates on posts, either with three red reflectors or markers with black and yellow strips at 45° towards the side of obstruction, meant to define obstructions or objects close to road. Object markers are circular red reflectors arranged on triangular or rectangular panels and are used to indicate hazard and obstructions within the path of vehicles, like the channelizing island placed close to the intersections.

Traffic islands

Traffic islands are raised areas constructed within the roadway to establish physical channels through which the vehicular traffic may be guided. Traffic islands often serve more than one function.

The traffic islands may be classified based on the function as :

(i) Divisional islands

(ii) Channelizing islands

(iii) Pedestrian loading islands

(iv) Rotary

Divisional islands are intended to separate opposing flow of traffic on a highway with four or more lanes. By thus dividing the highway into two one-way roadways, the head-on collisions are eliminated and in general other accidents are also reduced. The width of the divisional islands should be large if the head light glare is to be reduced during night driving. The kerb should be high enough to prevent vehicles from entering into the islands.

Channelizing islands are used to guide the traffic into proper channel through the intersection area. Channelizing islands are very useful as traffic control devices for intersection at grade, particularly when the area is large. The size and shape of the channelizing islands will very much depend upon the layout and dimensions of the intersections. Considerable professional experience and skill is required for the successful design of channelizing islands. If the islands are not properly designed and placed, there is a possibility of violation of rules be the traffic resulting in greater hazards. The various uses of properly designed channelizing islands are listed below :

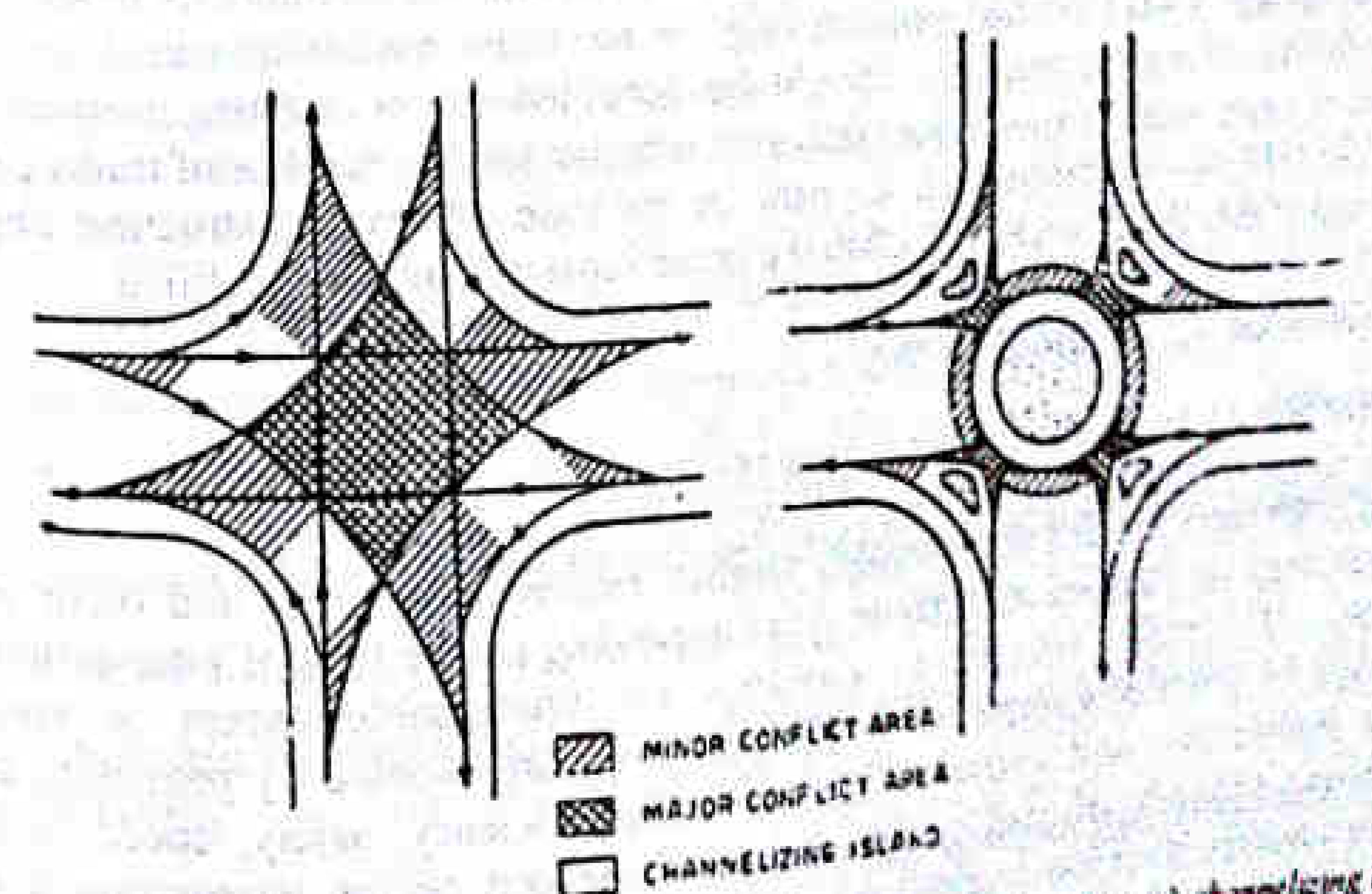
(i) The area of possible conflicts between traffic stream is reduced. This is illustrated in Fig. 5.32. By introducing channelizing islands both the major and minor conflict areas are reduced.

(ii) They establish the desired angles of crossing and merging of traffic streams.

(iii) They are useful when the direction of the flow is to be changed.

(iv) They serve as convenient locations for other traffic control devices.

(v) They serve as refuge islands for pedestrians.



(a) Area of conflict without channelizing island *(b) Area of conflict with channelizing islands*

Fig. 5.32 Conflict Areas

The design and functions of *rotary islands*, has been discussed in detail in Art. 5.4.1 under traffic rotary.

Pedestrian loading islands are provided at regular bus stops and similar places for the protection of passengers. A pedestrian island at or near a cross walk to aid and protect pedestrian crossing the carriageway may be termed as pedestrian refuge islands. For crossing multilane highways, pedestrian refuge island after two or three lanes would be desirable. The area in the roadway adjacent to the kerb which is kept reserved for use by stopped bus may be called a bus kerb loading zone.

Rotary island is the large central island of a rotary intersection; this island is much larger than the central island of channelized intersection. The crossing manoeuvre is converted to weaving by providing sufficient weaving length. Further details are given under Rotary Intersection in Art. 5.4.2.

5.3.3 Control of Access on Highways

If effective access control is not affected along a highway facility, ribbon development and encroachments follow, resulting in increase in the number of accidents and considerable reduction in level of service for vehicle operation. The control of access can either be full or partial. Full control of access on highways means that the authority to control the access is exercised to give preference to through traffic by providing access connections with selected public roads only and by prohibiting crossings at grade or direct private drive way connection. When there may be some private drive way connections and some crossings at grade, this is called partial control of access.

Express ways are divided arterial highways for motor traffic with full or partial control of access and generally provided with grade separation at intersection. Arterial highways are primarily meant for through traffic, usually on a continuous route and have partial control of access.

Major corridors of inter-city traffic are increasing in importance and are to be protected from unregulated road side development by exercising limited access control.

Grade separation across highways may be provided at intersections of divided rural highways, if the AADT of fast vehicles only on the cross road within next five years exceeds 5000. Grade separation should be provided across existing railway level crossings, if the product of AADT of fast road vehicles and the number of trains per day exceeds 50,000 within the next five years; in the case of new construction like bye passes, even if this figure exceeds 25,000 the grade separation may be justified.

5.4 DESIGN OF INTERSECTION

5.4.1 General

At the intersection there are through, turning and crossing traffic and these traffic movements may be handled in different ways depending on the type of intersection and its design. Generally intersection problems are unavoidable except in case of expressways or freeway systems where such problems are avoided by providing grade separated intersection and controlled access. The efficiency, safety, speed, cost of operation and capacity of road system very much depend on the intersection design. Pedestrian movements at intersection procedure increased produce hazards and delays.

Intersections may be classified into two broad groups :

(i) *Intersection at grade* : These include all roads which meet at more or less the same level. The traffic manoeuvres like merging, diverging and crossing are involved in the intersections at grade.

(ii) *Grade separated intersection* : The intersecting roads are separated by difference in level, thus eliminating the crossing manoeuvres.

Some of the traffic factors to be considered in intersection design are relative speed and maneuver areas.

Relative speed is an important factor in traffic flow at grade. Relative speed is the vector difference in the velocities of two vehicles in the same flow and is the sum of the speeds of approaching vehicles from opposite direction. It is the speed of convergence of vehicles in separate traffic flows as they approach a point of potential collision. Relative speed is dependent on the absolute speeds of the intersecting vehicles and the angles between them. When the angle of merging is small, the relative speed will also be low. If there is a collision between two vehicles at small angle at about the same speed or at low relative speed, the impact would be much less than when vehicle collide at high relative speed. As the relative speed increases, the judgement of drivers regarding time and distance is likely to be more inaccurate, thus increasing the possibility and severity of accidents. Thus in intersection design care has to be taken to keep the relative speed low.

Manoeuvre areas are those areas where, in actual manoeuvre, there is a potential collision and also those like channels of approach and departure where the manoeuvre is influenced. *Elemental manouvre areas* are those formed by only two single one-way lanes of flows when they diverge, merge or cross, these being the simplest of these manoeuvres. But in *multiple manoeuvre areas* where more than two one-lane one-way flows are present, traffic operations are much more complex and hence are to be avoided in intersection design. The point where the possible path of two vehicles intersect is called conflict points and the area containing all possible conflict points is the conflict area. In intersection design the conflict area, especially the major conflict area where more than one vehicle is subjected to conflict simultaneously, should be minimum.

5.4.2 Intersection at Grade

All road intersections which meet at about the same level allowing traffic manoeuvres like merging, diverging, crossing, and weaving are called intersections at grade. These intersections may be further classified as unchannelized, channelized and rotary intersections.

The basic requirements of intersection at grade are :

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

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

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

(iv) Sudden change of path should be avoided.

(v) Geometric features like turning radius and width of pavement should be adequately provided.

(vi) Proper signs should be provided on the road approaching intersection to warn the drivers.

(vii) Good lighting at night is desirable.

(viii) If the number of pedestrians and cyclists are large, separate provision should be made for their safe passage in intersections with high volume of fast moving traffic.

The various forms of intersections are shown in Fig. 5.33.

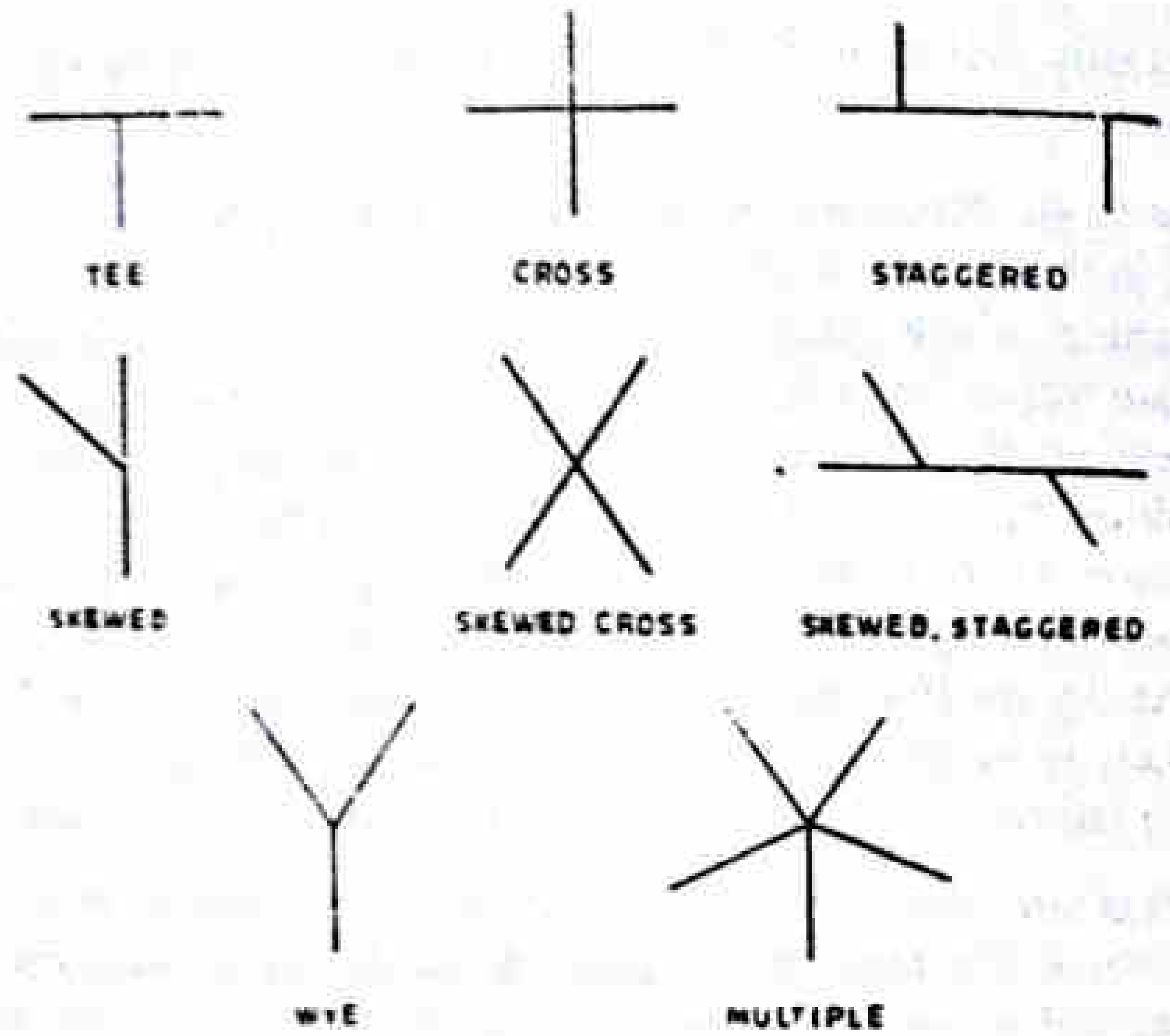


Fig. 5.33 Forms of Intersections

Unchannelized intersections

The intersection area is paved and there is absolutely no restriction to vehicles to use any part of intersection area. Hence the unchannelized (all-paved) intersections are the lowest class of intersection, easiest in the design, easiest in the design; but most complex in traffic operations resulting in maximum conflict area and more number of accidents, unless controlled by traffic signals or police. When no additional pavement width for turning movement is provided, it is called plain intersection. But when the pavement is widened at the intersection area, by a traffic lane or more, it is known as *flared* intersection. These have been illustrated in Fig. 5.34 alongwith common types of unchannelized intersections. The arrows indicate the path of traffic flow, turning, crossing and through movements. It may be seen that the conflict area is quite large as path of turning vehicles are not restricted or controlled. One of the crossing vehicles will have to stop while the other proceeds.

Channelized Intersections

Channelized intersection is achieved by introducing islands into the intersectional area, thus reducing the total conflict area available in the unchannelized intersection. The radius of the entrance and exit curves and the area are suitably designed to accommodate the channelizing islands of proper size and shape. These islands help to channelized turning traffic, to control their speed and angle of approach and to decrease the conflict area at the intersection. Some of the typical examples of channelized intersection are shown in Fig. 5.35. Channelization may be either partial or complete with *divisional* and *directional* islands and medians. From traffic operation point of view there is a better control on the traffic entering and leaving the intersection and hence channelized intersections are considered superior to the all-paved types. However one of the crossing vehicles will have to stop while the other proceeds.

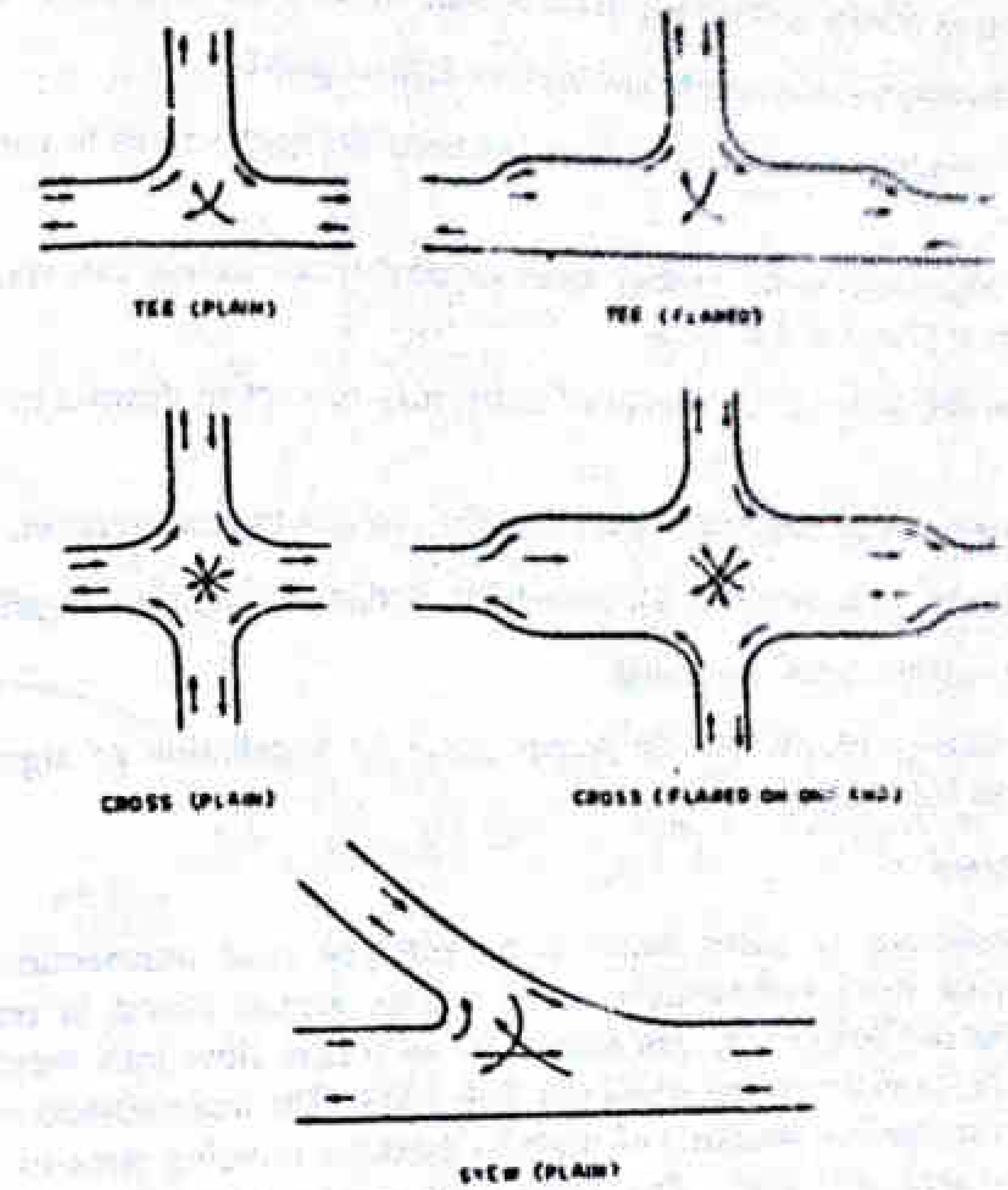


Fig. 5.34 Unchannelized Intersections

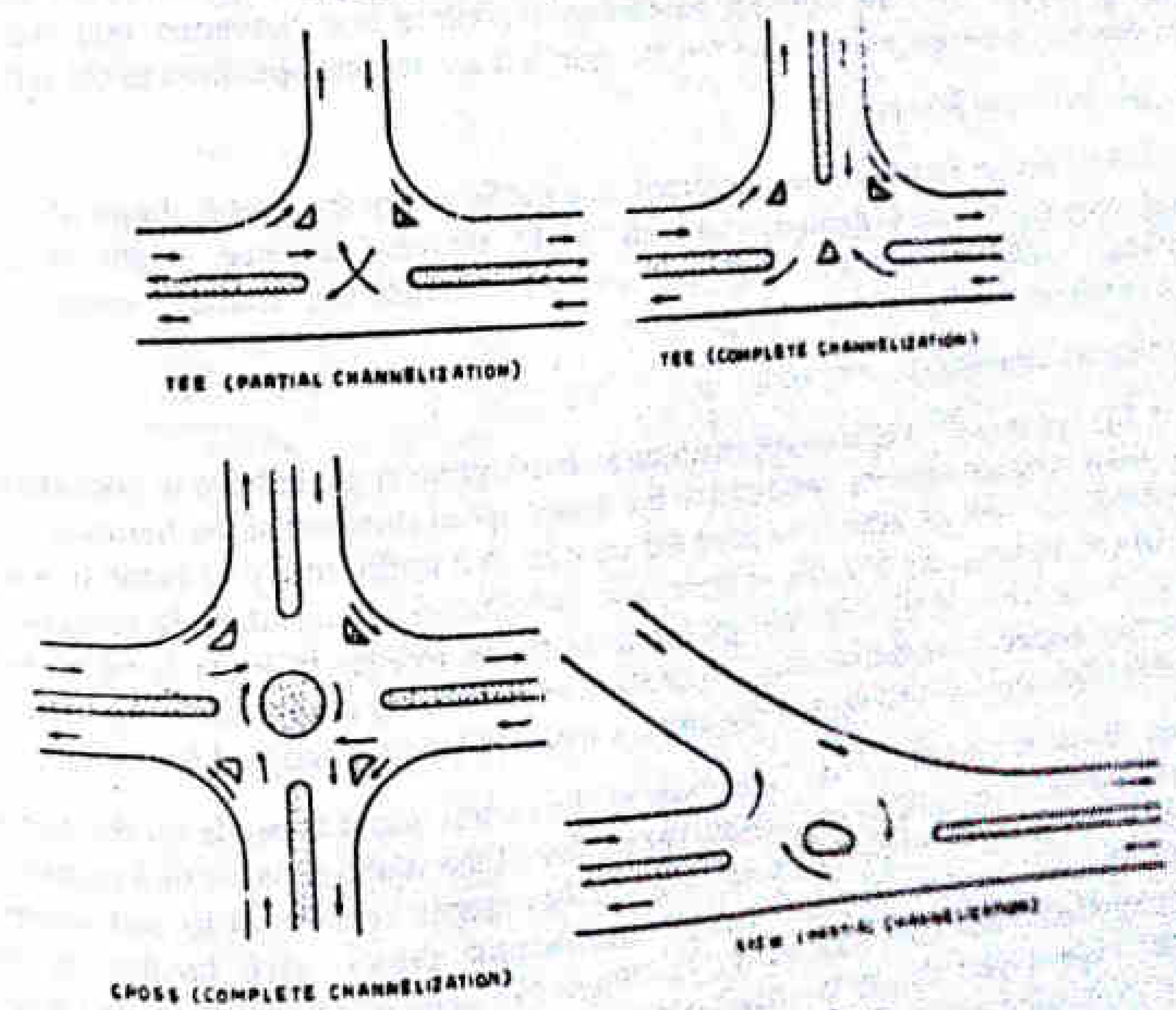


Fig. 5.35 Channelized Intersections

The advantages of channelized intersections may be summed up as follows :

(i) By canalization vehicles can be confined to definite paths.

(ii) Angle of merging streams can be forced to be at flat angles so as to cause minimum disruption.

(iii) Both the major and minor conflict areas within the intersection can considerably be decreased, as shown in Fig. 5.32.

(iv) Angle between intersecting streams of traffic may be kept as desired in a favourable way.

(v) Speed control can be established over vehicles entering the intersection.

(vi) Refuse islands can be provided for pedestrians within the intersection area.

(vii) Points of conflicts can be separated.

(viii) The channelizing islands provide proper place for installation of signs and other traffic control devices.

Rotary intersection

A rotary intersection or traffic rotary is an enlarged road intersection where all converging vehicles are forced to move round a large central island in one direction (clock wise direction) before they can weave out of traffic flow into their respective directions radiating from the central island (see Fig. 5.36). The main objects of providing a rotary are to eliminate the necessity of stopping even for crossing streams of vehicles and to reduce the area of conflict. The crossing of vehicles is avoided by allowing all vehicles to merge into the streams around the rotary and then to diverge out to the desired radiating road. Thus the crossing conflict is eliminated and converted into weaving manoeuvre or a merging operation from the right and a diverging operation to the left.

Design Factors of Rotary

Various design factors to be considered in a traffic rotary are speed, shape of central island, radius of rotary roadway, weaving angle, weaving distance, width of rotary roadway, radius of entrance and exit curves, channelizing islands, camber and superelevation, grade, lighting and signs.

These are briefly explained here.

(i) *Design speed* : Vehicles approaching as intersection at grade have to considerably slow down their speed when compared to the design speed standard of the highway under consideration. Though there is no need for vehicles in a traffic rotary to come to a dead stop before allowing cross traffic to cross, still there has to be considerable reduction in speed. With these in view the design speed for traffic rotaries in India is taken as 40 kmph for rotaries in rural area when one or more of converging roads is/are important. In all other cases and for rotaries in urban areas, a speed 30 kmph is adopted for design.

(ii) *Shape of central island* : The shape of the central island depends on the number and the layout of the intersecting roads. The outline of the island consists of a number of curves of large radii, without corners. The various shapes considered to suit different conditions are circular, elliptical, turbine and tangent shapes, each having its own advantages and limitations (see Fig. 5.37). When two equally important roads cross at roughly right angles i.e., all the four radiating roads placed symmetrically, a circular

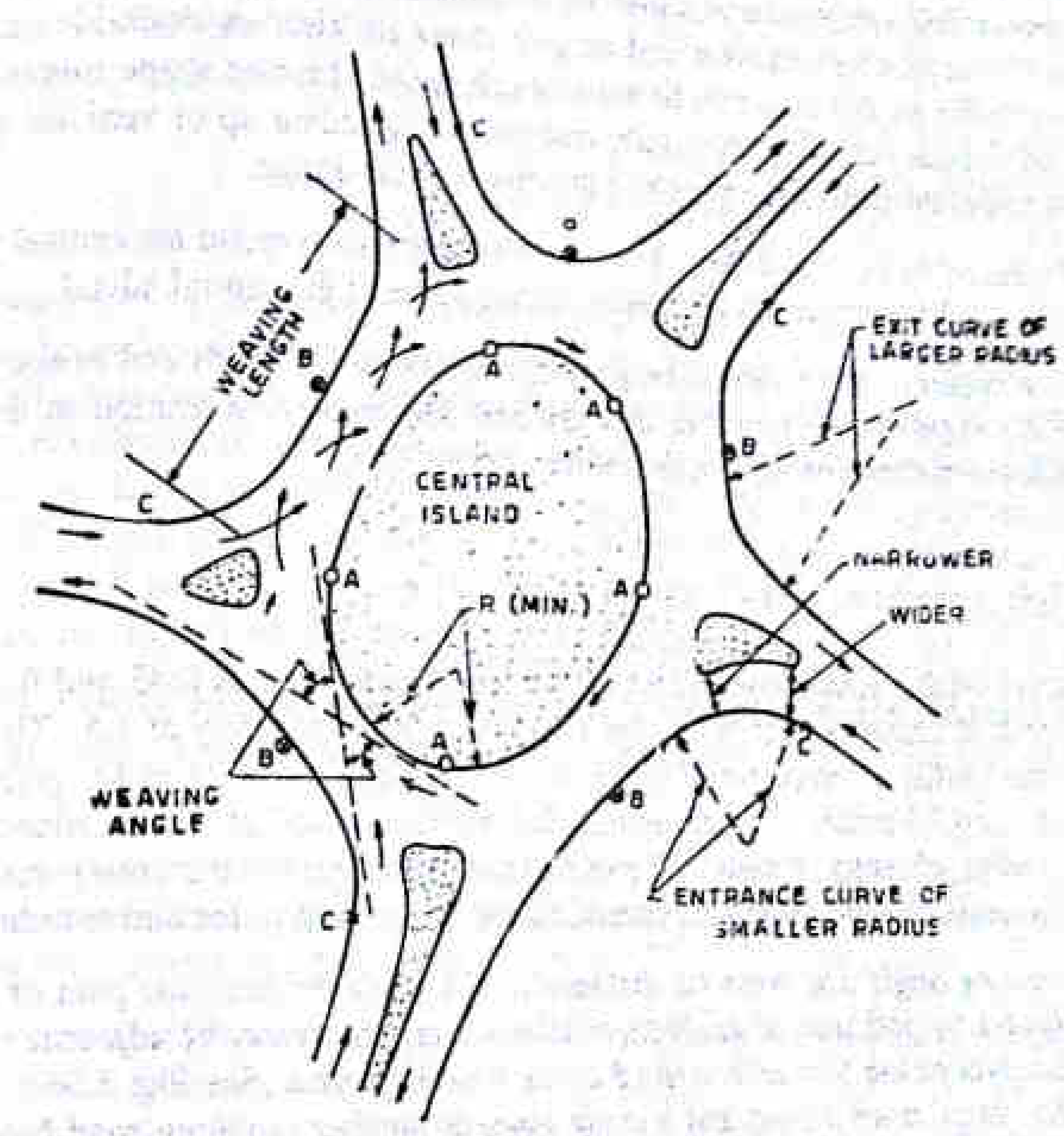


Fig. 5.36 Rotary Intersection

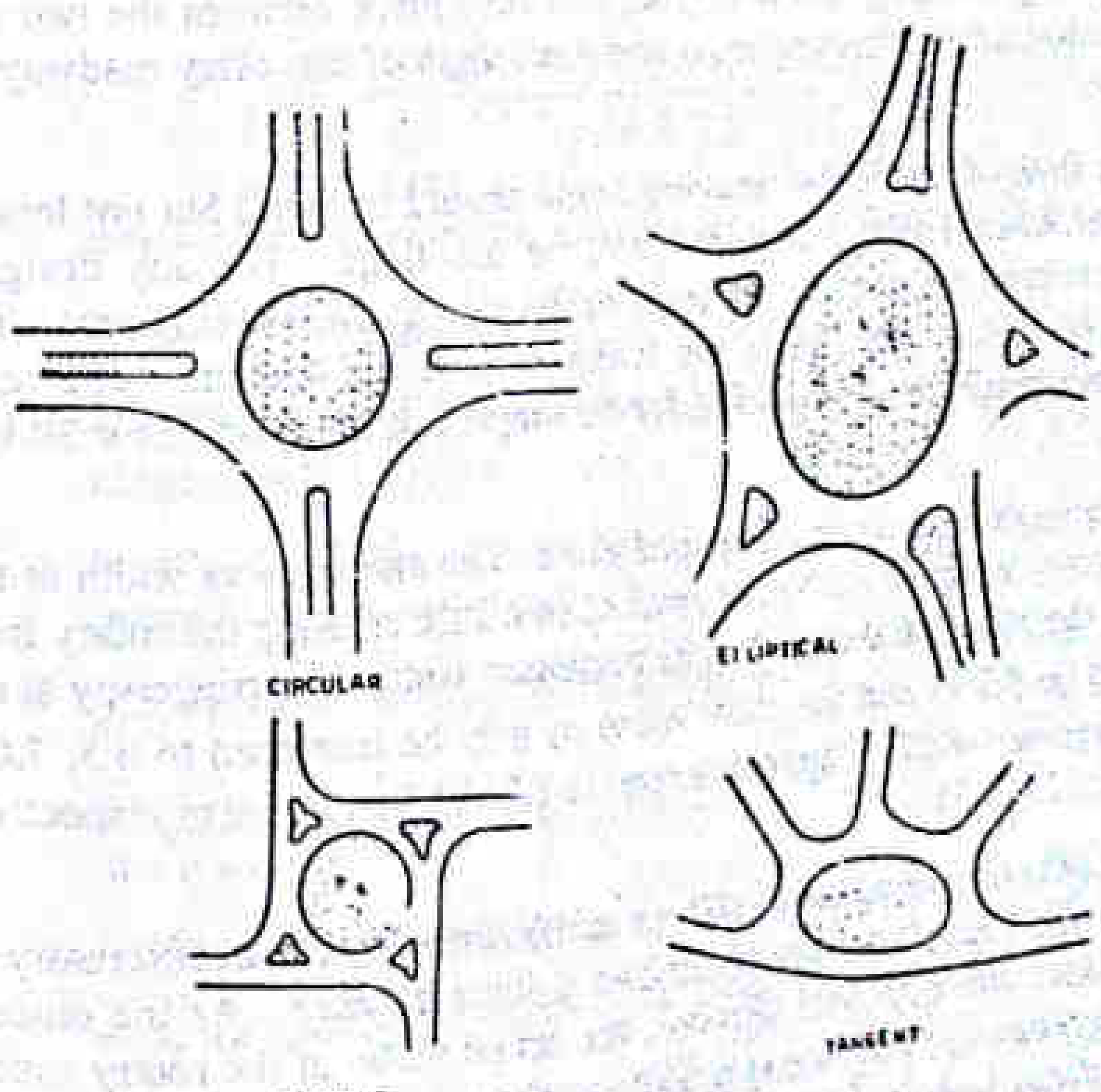


Fig. 5.37 Shapes of Rotary Islands

shape is suitable. The island may be often elongated to accommodate in the layout four or more intersecting roads; and to allow for the greater traffic flow along the direction of elongation. Two much elongation and tangent shape are also not desirable as there is a tendency of traffic in this direction to move much faster. Turbine shape forces reduction in speeds of vehicles entering the rotary and enables speeding up of vehicles going out, however at night, the head light glare is a limitation of the design.

(iii) *Radius of rotary roadway* : The one-way rotary road round the central island has different radii at different points depending on the shape of the central island.

Adequate superelevation cannot be provided on the rotary roads and hence it is safer to neglect the superelevation and to take friction only into consideration in the Eq. 4.8 and 4.9 to arrive at the allowable radius of the curve,

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

The values of the design coefficient of friction 'f' are taken as 0.43 and 0.47 for the speeds 40 and 30 kmph respectively, after allowing a factor of safety of 1.5. The IRC has suggested the radius of entry curve to be 20 to 35 m and 15 to 25 m for rotary design speeds of 40 and 30 kmph. The recommended minimum radii of central island are 1.33 times the radius of entry curves. Though these radii are for the rotary roadway, in practice it is convenient to design the central island to conform to the above radii.

(iv) *Weaving angle and weaving distance* : The angle between the path of a vehicle entering the rotary and that of another vehicle leaving the rotary at adjacent road, thus crossing the path of the former is termed as the weaving angle (See Fig. 5.36). Vehicles entering the rotary from a road and leaving towards another radiating road have to first merge into the one-way traffic flow in the rotary roadway around the central island and then weave out to diverge from this flow to the required road outlet. The weaving operation including merging and diverging can take place between the two channelizing islands of the adjacent intersecting legs, and this length of the rotary roadway is known as *weaving length.*

For smooth flow of traffic the weaving angle should be small but not less than 15° as the diameter of central island required will be too large. For any design speed the freedom of movement on a rotary depends on the size of the weaving area. The weaving length should be atleast four times the width of weaving section. The recommended value of weaving length are 45 to 90 m for 40 kmph and 30 to 60 m for 30 kmph design speeds.

(v) *Width of carriageway at entry and exit* : The carriageway width at the entrance and exit of a rotary is governed by the amount of traffic entering the rotary from the road or that leaving the rotary to the road. The minimum width of carriageway at the entrance and exit should be 5.0 m and the entry width e_1 may be increased to 6.5, 7.0 and 8.0 m when the carriageway width of approach road is 7.0, 10.5 and 14.0 m respectively and the radius at entry is 25 to 35 m.

(vi) *Width of rotary roadway* : All the traffic entering the traffic rotary have to go round the one-way rotary roadway for atleast a short distance. As the outer kerb lines follow the entrance and exit-sides of roads, the actual width of the rotary roadway varies from section to section. The minimum width of the roadway between edge of the central island and adjoining kerb is the effective width of the rotary roadway or of the weaving section and this by and large determines the capacity of the rotary.

The width of non-weaving section e_2 of the rotary should be equal to the widest single entry to the rotary and should generally be less than the width of weaving section. The width of weaving section W of the rotary should be one traffic lane wider than the mean width of the entry and non-weaving section i.e.;

$$W = \left[\frac{(e_1+e_2)}{2}+3.5\right] \text{m} \qquad (5.26)$$

(vii) *Entrance and exit curves* : The curve traced by the inner rear wheel of vehicles determines the radius and shapes to which the kerb line is to be set. A vehicle entering a rotary has to slow down to the design speed of the rotary and therefore the radius of the entrance curve should be the same as the minimum recommended radius of the central island. For the design speed of 40 kmph the suggested radius at entry curves is 20 to 35 m and for 30 kmph, 15 to 25 m. It has been seen that the buses and trucks can take right angled turn easily at these curves at the design speeds. Where practicable three centred entry curves may be provided instead of simple circular curve.

Vehicles leaving the rotary would accelerate to the speed of the radiating roads and hence the exit curves should be of a larger radius than entry curves; one and a half to two times radius of entry is considered reasonable.

The normal pavement width at entrance and exit should be equivalent to two lanes in order to prevent clustering of mixed traffic at the approaches. Extra widening has to be provided at the entrance and exit curve.

The pavement width at entrance curve will be higher than at exit curve as the radius of the former is less than the latter.

(viii) Capacity of the rotary : The practical capacity of the rotary is dependent on the minimum capacity of the individual weaving section. The capacity is calculated from the formula :

$$Q_P = \frac{280\,W\,(1+e/W)(1-p/3)}{(1+W/L)} \qquad (5.27)$$

where Q_P = practical capacity of the weaving section of a rotary in pcu per hour.

W = width of weaving section (6 to 18 m)

e = average width of entry e_1 and width of non-weaving section e_2 for the range e/W = 0.4 to 1.0

L = length of weaving section between the ends of channelizing islands in metre for the range of W/L = 0.12 to 0.4

p = proportion of weaving traffic given by

$$p = \frac{b+c}{a+b+c+d} \text{ in the range 0.4 to 1.0}$$

a = left turning traffic moving along left extreme lane

d = right turning traffic moving along right extreme lane

b = crossing/weaving traffic turning towards right while entering the rotary

c = crossing/weaving traffic turning towards left while leaving the rotary

Some corrections have been suggested in the calculated capacity values depending on the entry, exit and internal angles and the pedestrian traffic in the rotary intersection. The IRC has recommended the following PCU values for finding the capacity of the rotary :

Cars, light commercial vehicles and three wheelers = 1.0

Buses, medium and heavy commercial vehicles = 2.8

Motor cycles, scooters = 0.75

Pedal cycles = 0.50

Animal drawn vehicles = 4 to 6

(ix) *Channelizing islands* : Channelizing islands should be provided at the entrance and exit of the rotary to prevent undesirable weaving, and turning and to reduce area of conflict. Further these channelizing islands help in forcing the vehicle to reduce their speed to the design speed of the rotary and to serve as convenient place for erecting traffic signs and as a pedestrian refuge. The shape and size of channelizing island is governed by the radius of the rotary the radii of the entrance and exit curves and the angles and layout of the radial road and rotary. The channelizing islands are generally provided with kerbs 15 to 21 cm high.

(x) *Camber and superelevation* : A vehicle passing along a rotary traverses a reverse curve while changing from one-way path of roadway to the exit of the radial road. Hence the cross slope of the rotary roadway at the point of change in direction should be minimum. The inward slope of the cross slope or camber serves as superelevation for the traffic going around the central island, though design of the curve has been made assuming no superelevation. The outer slope of the camber helps the vehicles turning left towards the exit curve to the radiating road.

(xi) *Sight distance, grade* : The sight distance in the rotary should be as large as possible and in no case less than the safe stopping distance for the design speed. The minimum sight distance should be 45 and 30 m for design speeds of 40 and 30 kmph respectively. It is preferable to locate a rotary on level ground. It may also be located on the area which is on a single plane, with the slope not exceeding 1 in 50 with the horizontal.

(xii) *Lighting* : The minimum lighting required is one each on the edge of central island facing each radiating road. (Points A in Fig. 5.36). Additional lights 'B' may be provided when the central island is larger than 60 m diameter. Light 'C' may also be provided near the entrance curve if the pedestrians are large in number.

(xiii) *Traffic signs* : The standard traffic (warning) signs indicating the presence of rotary intersection should be installed at all approaching roads to give advance information to traffic. At night a red reflector or red light is placed at about one metre above the road level on the nose of each directional island and on the kerb of the central island facing the approaching roads. Vertical black and white strips of width 25 to 30 cm painted on kerb of central island and channelizing islands improve visibility.

(xiv) *Provision for cyclists and pedestrians* : One of the main use of traffic rotary of non-stop and consistent journey is lost, if pedestrians are allowed to enter the rotary intersection or if pedestrian crossings are provided and vehicles are controlled by stop signals. Also the rotary would become a constant problem for traffic control and enforcement. Hence as far as possible pedestrians and even cyclists should be isolated from the general traffic utilizing the rotary. In India the problem is very typical as rotaries are needed in urban areas where the number of pedestrians and cyclists are also high, making the problem complex. If the number of cyclists are less than 50 per hour, they may be permitted to mix up with the other traffic using the rotary ; if they are more, a separate cycle track to segregate cyclists will be desirable. If there are a large number of pedestrians, separate foot path with guard rails should be provided around the rotary on the outer side to prohibit them from entering the rotary. However, if they are allowed to cross along the pedestrian crossing near the channelising islands, there would be problems of stopping the stream of fast vehicles entering and leaving the rotary. Provision of crossing facilities to pedestrian by *subway* or *over bridge* is possible solution, but the proposal would however be costly.

Conditions when traffic rotary is justified

Construction of a traffic rotary needs large area which may be available in rural areas at reasonable cost. But in India generally the volume of fast moving traffic is very low in rural areas. There are various other points to be considered before the construction of a traffic rotary can be justified.

The American Association of State Highway Officials, now *AASHTO* have suggested that the lowest limit of traffic volume when a traffic rotary is justified is bout 500 vehicles per hour on all intersecting roads put together and the maximum limit beyond which rotary may not efficiently function is about 5000 vehicles per hour. However, if a large proportion of traffic is turning traffic, provision of rotary even outside these limits is justified.

However the IRC suggests that the maximum volume of traffic that a rotary can efficiently handle is 3000 vehicles per hour entering from all the legs of the intersection.

Keeping in view the mixed traffic conditions, it is recommended by the Indian Roads Congress that traffic rotaries may be provided where the intersecting motor traffic is about 50 percent or more of the total traffic on all intersecting roads or where the fast traffic turning right is as least as 30 percent of the total traffic.

Advantages and limitations of traffic rotary

Various Advantages of Rotary

(i) Crossing manoeuvre is converted into weaving or merging and diverging operations. Hence there is no necessity of any of the vehicles, even those which have to go in cross directions, to stop and proceed within a traffic rotary. Thus the journey is more consistent and comfortable when compared with any other intersection at grade.

(ii) All traffic including those turning right or going straight across the rotary have equal opportunity as those turning left.

(iii) The variable cost of operation of automobile is less at a traffic rotary than at a signalized intersection where the vehicles have to stop and proceed. Though the distance to be traversed by vehicles which are to turn to the right or proceed straight across is higher, still the fuel consumed in the process of crossing the rotary intersection is likely to be less. This is because one stop-proceed operation at a signal is likely to consume fuel required for travelling about 275 metre at a uniform speed without stopping.

(iv) There is no necessity of traffic police or signal to control the traffic as the traffic rotary could function by itself as a traffic controlled intersection and is the simplest of all controls. The maintenance cost is hence almost nil.

(v) The possible number of accidents and the severity of accidents are quite low because of low relative speed. Further weaving, merging and diverging manoeuvres are easier and less dangerous operation than crossing. Check on speed of vehicles is automatically enforced by proper design.

(vi) Rotaries can be constructed with advantage when the number of intersecting roads is between four and seven.

(vii) The capacity of the rotary intersection is the highest of all other intersections at grade. The rotary can accommodate a total traffic upto about 3000 vehicles per hour and enable radial streets to carry traffic almost to their full capacity.

Various Limitations of Rotary

(i) Rotary requires comparatively a large area of land and so where space is limited and costly as in built up areas, the total cost may be very high.

(ii) Where pedestrian traffic is large as in urban areas the rotary by itself cannot control the traffic and hence has to be supplemented by traffic police. If the vehicular traffic have to stop to allow pedestrian to cross, the main purpose of rotary is defeated.

(iii) In places where there is mixed traffic and large number of cyclists and pedestrians, the design of rotary becomes too elaborate and operation and control of traffic also become complex.

(iv) Where the angle of intersection of two roads is too acute or when there are more than seven intersecting roads, rotaries are unsuitable.

(v) When the distance between intersections on an important highway is less, rotaries become troublesome.

(vi) Where there are a large number of cycle and animal drawn vehicles, the extra length to be traversed by crossing and right turn traffic is considered troublesome and there is a tendency to violate the traffic regulation of clock wise movement around the central island.

(vii) When the traffic volume is very low as in most of the rural areas of India, construction of a rotary cannot be justified.

5.4.3 Grade Separated Intersections

Grade separated intersection design is the highest form of intersection treatment. This type of intersection causes least delay and hazard to the crossing traffic and in general is much superior to intersections at grade from the point of view of traffic safety and efficient operation.

A highway grade separation is achieved by means of vertical level. Separation of intersecting roads by means of a bridge thus eliminating all crossing conflicts at the intersection. The grade separation may be either by an over bridge or under pass Transform of route at the grade separation is provided by interchange facilities consisting of ramps. Interchange ramps may be classified as direct, semi-direct or indirect as shown in Fig. 5.38. The direct interchange ramp involves diverging to right side and merging from the right. Semi-direct interchange ramp allows diverging to left but merging is from right side. In the indirect method of interchange ramp, a simple diverging to the left and a merging from the left side are involved which are simpler and less hazardous than diverging to the right and merging from right; but the distance to be traversed in indirect interchange is more.

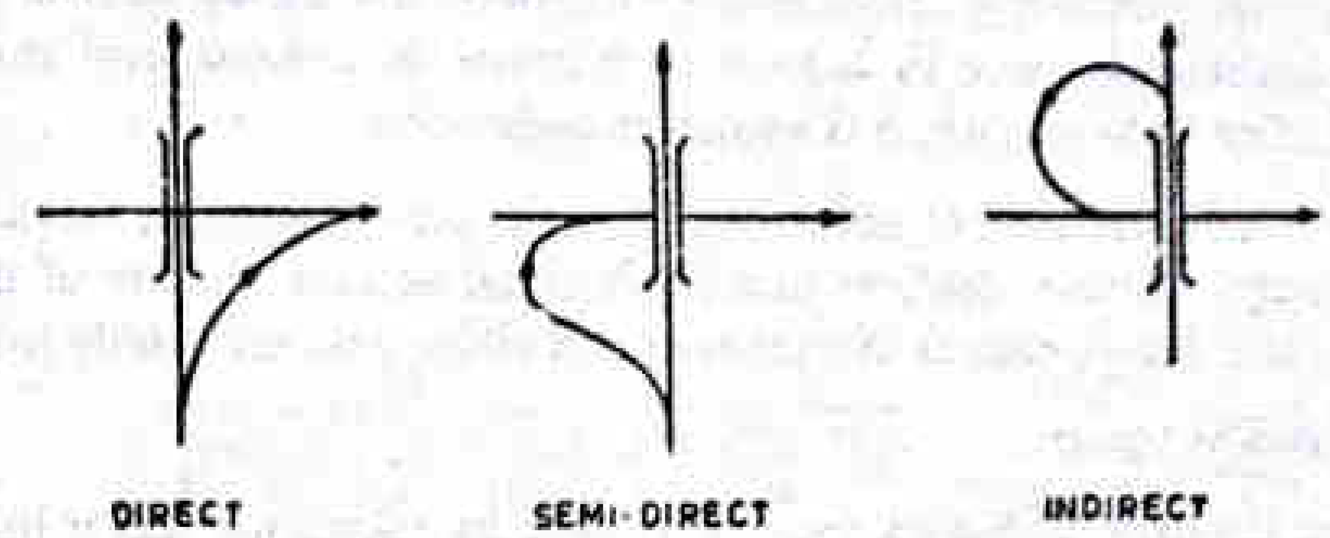


Fig. 5.38 Interchange Ramps

The grade separated intersections have the following advantages and limitations.

Advantages of Grade Separation

(i) Maximum facility is given to the crossing traffic. As the roads are separate, this avoids necessity of stopping and avoids accidents while crossing.

(ii) There is increased safety for turning traffic and by indirect interchange ramp even right turn movement is made quite easy and safe by converting into diverging to left and merging from left.

(iii) There is overall increase in comfort and convenience to the motorists and saving in travel time and vehicle operation cost.

(iv) The capacity of the grade operated intersection can practically approach that of the two cross roads.

(v) Grade separation is an essential part of controlled access highway like expressway and freeway.

(vi) It is possible to adopt grade separation for all likely angles and layout of intersecting roads.

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

Disadvantages of Grade Separation

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

(ii) Where there is limited right of way like built up or urban area or where the topography is not favourable, construction of grade separation is costly, difficult and undesirable.

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

Grade separation structures

The various types of bridge structures used to separate the grades of the two intersecting highway may be T-beam bridge arch bridge, rigid portal frame type and prestressed concrete bridges. There should be vertical clearance of atleast 4.3 m and if double decked vehicles are anticipated, the clearance should be 5.2 metre. The type of the bridge structure should be selected depending upon the design, construction and other considerations like site conditions and aesthetics.

The grade separated intersections are classified as *over-pass* and *under-pass*. When the major highway is taken above by raising its profile above the general ground level by embankment and an overbridge across another highway, it is called an over-pass. On the contrary if the highway is taken by depressing it below the ground level to cross another road by means of an under-bridge, it is known as under-pass.

The choice of the over-pass or under-pass depends on topography, vertical alignment, drainage, economy, aesthetic features and preferential aspects for one of the highways. The advantages and disadvantages of over-pass and under-pass are briefly listed below.

Advantages of an Over-pass

Troublesome drainage problems may be reduced by taking the major highway above the cross road. For the same type of structure when the wider road is taken above the span of the bridge being small, the cost of the bridge structure will be less. In an over-pass of major highway, there is an aesthetic preference to the main through traffic and less feeling of restriction or confinement when compared with the under-pass. Future expansion or lateral expansion or construction of separate bridge structure for divided highway is possible.

Disadvantages of an Over-pass

In rolling terrain if the major road is to be taken above, the vertical profile will also have rolling grade line. If the major highway is to be taken over by constructing high embankments and by providing steep gradients, the increased grade resistance may cause speed reduction on heavy vehicles. Also there will be restrictions to sight distance unless long vertical curves are provided.

Advantages of an Under-pass

There is a warning to traffic in advance due to the presence ot an under pass which can be seen from distance. When the major highway is taken below, it is advantageous to the turning traffic because the traffic from the cross road can accelerate while descending the ramp to the major highway and the traffic from the major highway can decelerate while ascending the ramp to the cross roads. The under-pass may be of advantage when the main highway is taken along the existing grade without alteration of its vertical alignment and cross road is depressed and taken underneath.

Disadvantages of an Under-pass

There may be troublesome drainage problems at the under pass, especially when the ground water level rises high during rainy season and the road at the under-pass is to be depressed as much as 5 m to 7 m below the ground level. It may be necessary even to pump water continuously during the period when water-logging problems exist. At under-pass the over head structure may restrict the vertical sight distance even at the valley curve near the under-pass. There is a feeling of restriction to the traffic at the sides while passing along the under-pass and unless the clearance is sufficiently large, this may affect the capacity at the intersection. There is no possibility of stage construction for the bridge structure at the under-pass.

Interchanges

Grade separated intersection with complete interchange facilities is essential to develop a highway with full control of access. When there is intolerable congestion and accidents at the intersection of two highways carrying very heavy traffic there is no better solution than to provide grade separated intersection.

Some of the types of interchanges are shown in Fig. 5.39. Of all these complete *clover leaf* fulfils all the requirements of turning traffic involving the simplest traffic maneuvers, viz. diverging to the left and merging from the left by providing four indirect ramps.

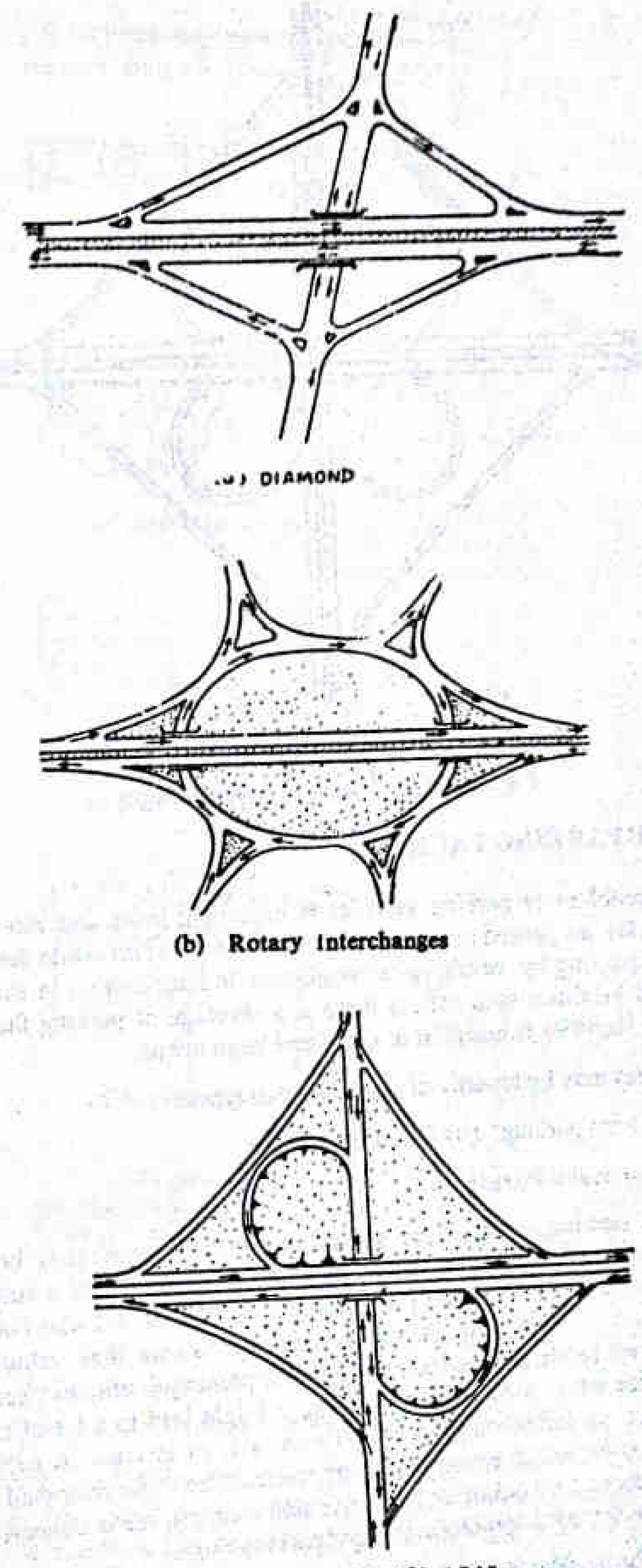


Fig. 5.39 (a) (b) & (c) Types of Interchanges (Contd.)

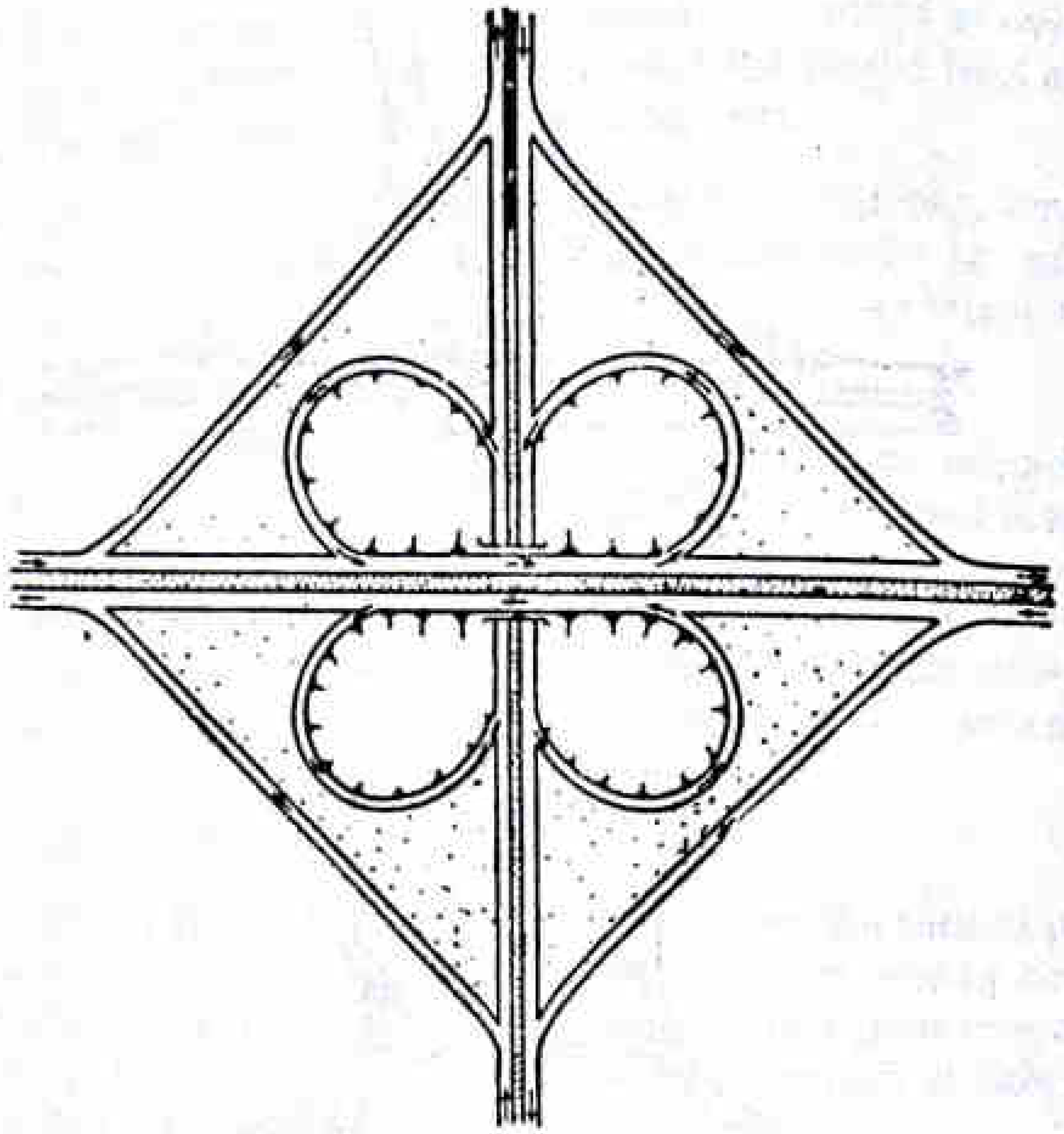


Fig. 5.39 (d) Types of Interchanges

5.5 DESIGN OF PARKING FACILITY

In cities the problem of parking vehicles is becoming more and more acute day by day. When vehicles are parked on the road side, even for a short while there is restriction to other vehicles passing by, resulting in congestion and accidents. In shopping centers, public places and localities with offices there is a shortage of parking facilities. Proper design of parking facilities is essential in cities and large towns.

Parking facilities may be broadly divided into two types :

(i) On-street or kerb parking, (ii) Off-street parking

5.5.1 On-Street or Kerb Parking

In this type of parking, vehicles are parked on the kerb which may be designed for parking. Kerb parking is quite convenient for those who could find a suitable space to park their vehicles near the place they wish to stop; but for others who could not find a parking space it is a problem and often they may have to park their vehicle at a far off place and walk down to the destination. Unless kerb parking facility has been adequately designed in advance while planning a new town, it might lead to a lot of inconvenience and congestion due to decreased road capacity as well as increase in accidents. Kerb parking facility may be either unrestricted or restricted type. The restricted kerb parking may either be controlled by police or by metres and a certain fee is collected from those who park their vehicles for a certain duration of parking time.

Angle parking or parallel parking may be allowed in the kerb parking. See Fig. 5.40. Angle parking may be at angles 30, 60 or 90 degrees. Angle parking accommodates

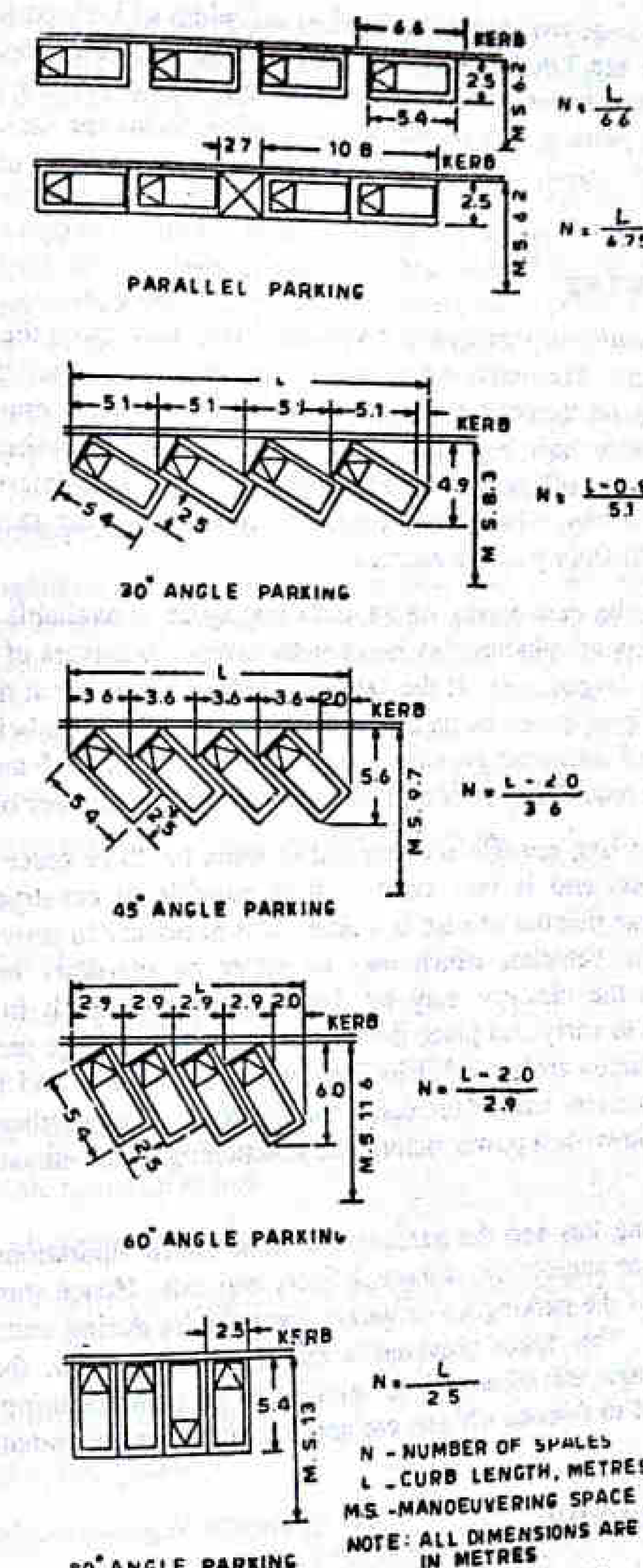


Fig. 5.40 Various Patterns of Kerb Parking

more vehicles per unit length of kerb and maximum vehicles that can be parked is with an angle of 90 degree. The width of road required for parking and unparking manoeuvre also is more with angle parking and it increases with the parking angle up to a maximum at 90° angle. Angle parking is more convenient for the motorists than the parallel parking, but it produces much more obstruction to the through traffic resulting in more accidents than the parallel parking. Out of various angles used in angle parking, 45 degree angle is considered the best from all considerations discussed above.

Parallel parking is generally preferred when the width of kerb parking space and the width of the street are limited. But the parking and unparking operations are more difficult needing a few forward and reverse movements before parking properly or before taking out. Parallel parking may be with equal spacing, facing the same direction or may be two cars placed closely with open intervals between two-car units, as shown in Fig. 5.40.

5.5.2 Off-Street Parking

When parking facility is provided at a separate place away from the kerb, it is known as off-street parking. The main advantage of this method is that there is no undue congestion and delay on the road as in kerb parking. But the main draw back is some of the owners will have to walk a greater distance after parking the vehicle. It is also not possible to provide the off-street parking facility at very close intervals especially in business centers of a city. Two basic types of off-street parking facilities are surface parking lots and multi-floor parking garages.

Parking lots may be convenient where sufficient space is available at comparatively low cost. The parking of vehicles may be done by owners or drivers of the cars and then this is called *self parking system*. If the vehicle is left by the driver at the entrance space and again collected from there, the parking and delivering operations being carried out by attendants, it is called *attendant parking system*. Most important advantage of attendant parking is less space required to store and manoeuvre the same number of cars.

Multi-storeyed parking garages are restored to when the floor space available for the parking garage is less and is very costly. It is possible to construct multi-storeyed garages to park a large number of cars at a time. It is necessary to provide the interfloor travel facility for the vehicles, which may be either by *elevators or by ramps*. In mechanized garages the elevator may be designed to move both in vertical and in horizontal directions to carry and place the vehicle in the appropriate parking stall and to deliver it back. If ramps are provided for driving the vehicles to and from the parking stall, the space requirement will be increased considerably. On the other hand, if there is a mechanical break down or a power failure, the functioning of the elevator system would come to a stand still.

Both in the parking lots and the garages, the basic traffic operations consist of five steps, namely, entrance acceptance, storage delivery and exit. Hence some definite space is required in front of the parking lot or garage for vehicles during entrance acceptance and exit operations. This space provided is called *reservoir area*, the size of which depends on the average rate of arrival of vehicles to be parked during peak hour, the average time required to dispose off one car and the number of attendants employed for storage-operations.

5.6 HIGHWAY LIGHTING

The rate of highway accidents and fatalities that occur during night driving is several times higher in terms of vehicle-kilometre, than that during day driving. One of the various causes of increased accident rate during night may be attributed to poor night visibility. Highway lighting is particularly more important at intersections, bridge site, level crossings and in places where there is restriction of traffic to movements. Lighting on rural roads has not yet become common, evidently due to the cost consideration and less number of pedestrians and other slow traffic using the facility at night. On urban

roads where the density of population is also high, road lighting has other advantages like feeling of security and protection. Thus even though head lights of vehicles may be sufficient for safe night driving, still road lighting may be considered as an added facility to the road users.

During night driving the manner in which objects are visible varies with both the absolute level of brightness and the relative brightness of the road surface and the object. When the brightness of the object is less than that of the background, that is when the object appears darker than the road surface, *discernment* is principally by *silhouette*. If the brightness of the pavement is uniformly increased, discernment by silhouette is enhanced. Hence it is obvious that night visibility on concrete and other light coloured pavements are better than on black top surfaces. A light coloured, rough textured pavement surface that can reflect light back is considered most desirable. Surface that becomes mirror like or shiny when wet (such as smoothened black top road surface) should be avoided as practically no light reflects back from them.

When the brightness of an object is more than that of the immediate background, discernment is by *reverse silhouette*. The objects adjacent to the roadway, projections above the pavement surface such as island or vehicles may be seen by this process of reverse silhouette. When the pavement surface is very dark like black top surface, the object which are relatively brighter in colour are seen by this process.

Thus the various factors that influence night visibility are :

(a) amount and distribution of light flux from the lamps.

(b) size of object.

(c) brightness of object.

(d) brightness of the background.

(e) reflecting characteristics of the pavement surface.

(f) glare on the eyes of the driver, and

(g) time available to see an object.

Design factors of highway lighting

Various factors to be considered in the design of road lighting are :

(i) Lamps

(ii) Luminaire distribution of light

(iii) Spacing of lighting units

(iv) Height and over hang of mounting

(v) Lateral placement

(vi) Lighting layouts

Lamps

The choice of the lamp, its type, size and colour depends on several considerations in addition to distribution of light flux on the pavement surface. It is economical to use the largest lamp size in a luminaire which will provide sufficient uniformity of pavement brightness; but this depends on the spacing of the lamps also. The various types of lamps in use for highway lighting are filament, fluorescent and sodium or mercury vapour lamps. The cheapest amongst these, is the filament lamp. Sodium-vapour lamps are preferred at large intersections.

Luminaire Distribution of Light

To have the best utility of the luminaire or source of light, it is necessary to have proper distribution of light. The distribution should be downward so that high percentage of lamp light is utilized for illuminating the pavement and adjacent area. The light distribution selected should be the one which would produce maximum uniformity of pavement brightness. The distribution from the luminaire should cover the pavement between the kerbs and provide adequate lighting on adjacent area i.e. 3 m to 5 m beyond the pavement edge. The illumination is necessary for traffic signs and other objects on the road.

There are five typical luminaire distributions (see Fig. 5.41) which meet most of the highway lighting requirements.

It is suggested that the average level of illumination on road side may be 20 to 30 lux on important urban roads carrying fast traffic and about 15 lux for other main roads carrying mixed traffic and in arterial roads. In secondary road it may be 4 to 8 lux depending on traffic. However the actual intensity of illumination in most of the existing roads may be lower than the above values.

The Indian Standards Institution recommends an average level of illumination of 30 lux on important roads carrying fast traffic and 15 lux on other main roads, the ratio of minimum to average illumination being 0.4.

Spacing of Lighting Units

The spacing of lighting units is often influenced by the electrical distribution poles, property lines, road layout and type of side features and their illumination. Large lamps with high mountings and wide spacings should be preferred from economy point of view.

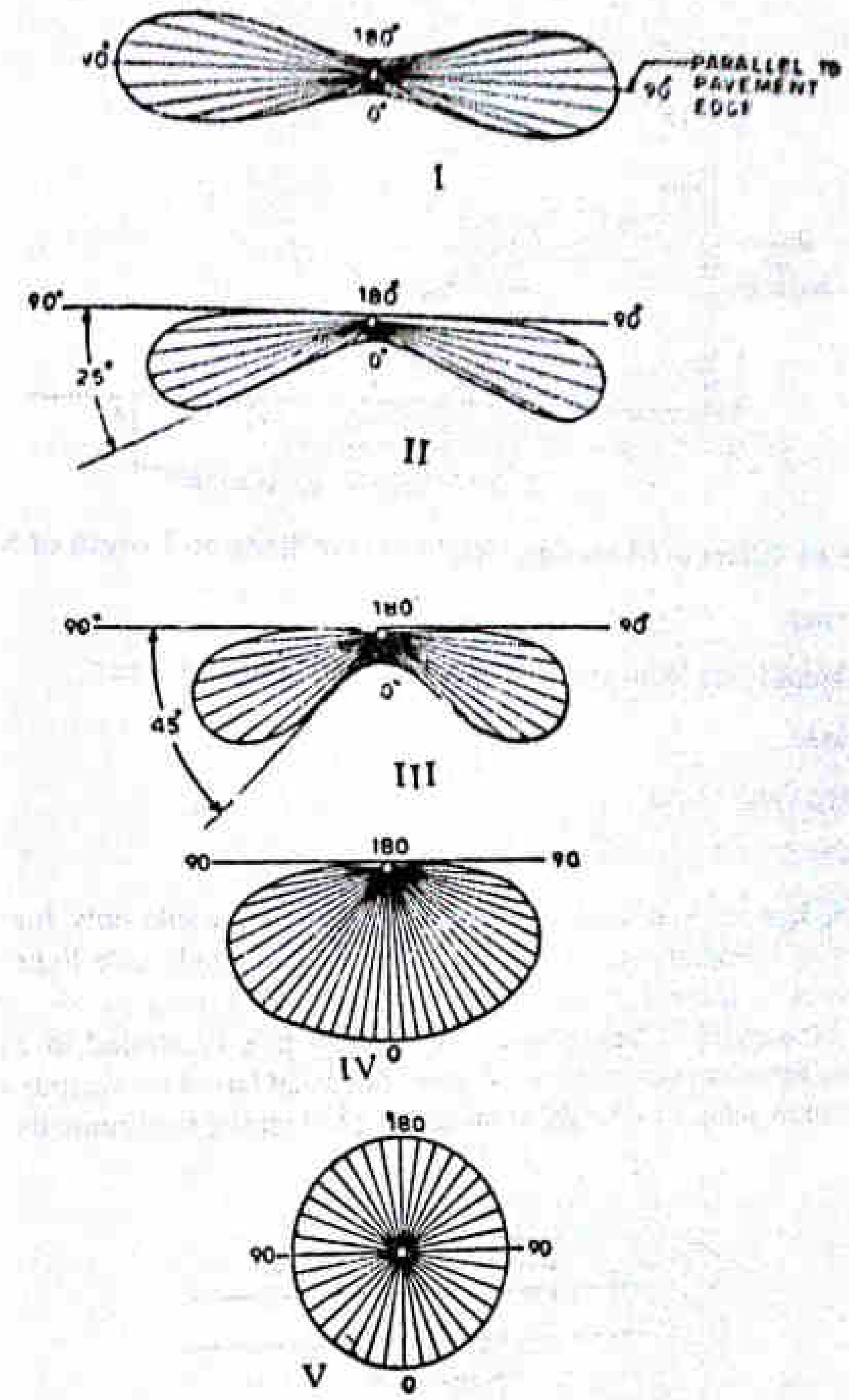
Height and Overhang of Mounting

The distribution of light, shadow and the glare effect from street lamps depend also on the mounting height. The glare on eyes from the mounted lights increases with the power of the lamp directed towards the eye and decreases with increase in height of mounting. Usual mounting heights range from 6 m to 10 m, higher values being preferred where possible at least for important urban roads. The minimum vertical clearance required for electric power lines upto 650 volts has been specified as 6 m above the pavement surface by the Indian Roads Congress.

Over hangs on the lighting poles would keep the poles away from the pavement edges, but still allow the lamp to be held above the kerb or towards the pavement. This enables better distribution of light on the pavement and less glare on eyes of road users. The effect of mounting height and over hang on the length of shadow is shown in Fig. 5.42. It is desirable to have higher mounting heights and necessary overhang projections.

Lateral Placement

The street lighting poles should not be installed close to the pavement edge. If they are too close to the carriageway, free movement of traffic is obstructed, decreasing the capacity of the roadway. Indian Roads Congress has specified the horizontal clearance required for lighting poles as given below :



I Two-way lateral distribution (for narrow roads)

II Narrow asymmetric lateral distribution (for narrow roads)

III Medium width asymmetric lateral distribution (for roads with medium width)

IV Wide asymmetric lateral distribution (for very wide highways)

V Normal symmetrical distribution (for mounting at centre of highways and at intersections)

Fig. 5.41 Types of Luminaire Distribution

(a)	For roads with raised kerbs (as in rural roads)	Minimum 0.3 m and desirable 0.6 m from the edge of raised kerb.
(b)	For roads without raised kerbs (as in rural roads)	Minimum 1.5 m from the edge of the carriageway, subject to minimum of 5.0 m from the centre line of the carriageway.

The clearance specified apply to poles carrying electric power and telecommunication lines also.

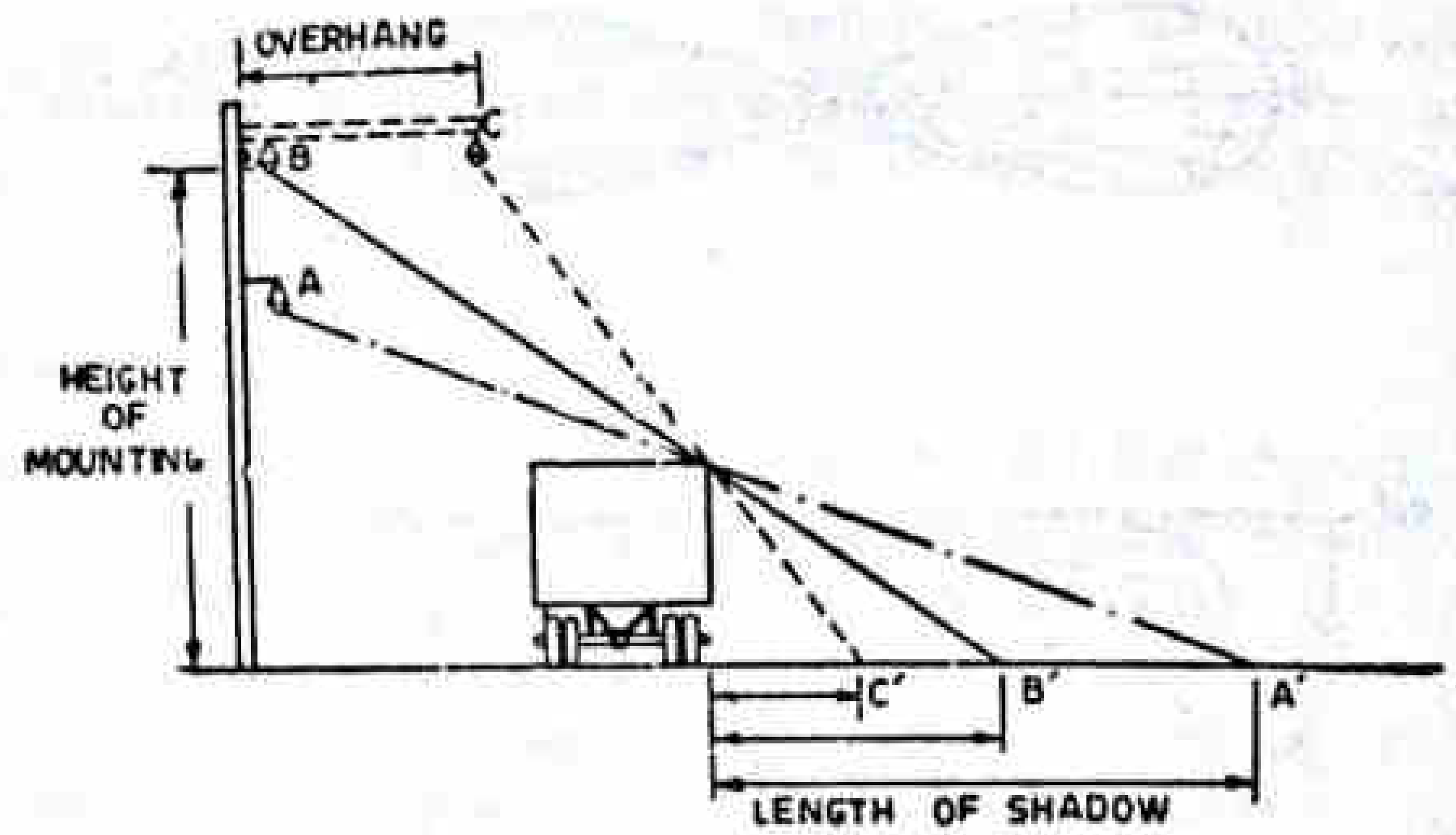


Fig. 5.42 Effect of Mounting Height of Overhang on Length of Shadow

Lighting Layouts

On straight roads the lighting layout may be of the following types :

(a) Single side

(b) Staggered (both sides)

(c) Central

Single side lighting is economical to install; but it is suitable only for narrow roads. Due to cost considerations even on two lane roads often single side lighting is adopted. For wider roads with three or more lanes the *staggered system* or the *central lighting system* may be adopted. These systems of lighting have illustrated in Fig. 5.43. The spacing of the lights in each of these systems is decided based on various considerations including location, lamp size, height of mounting and lighting requirements.

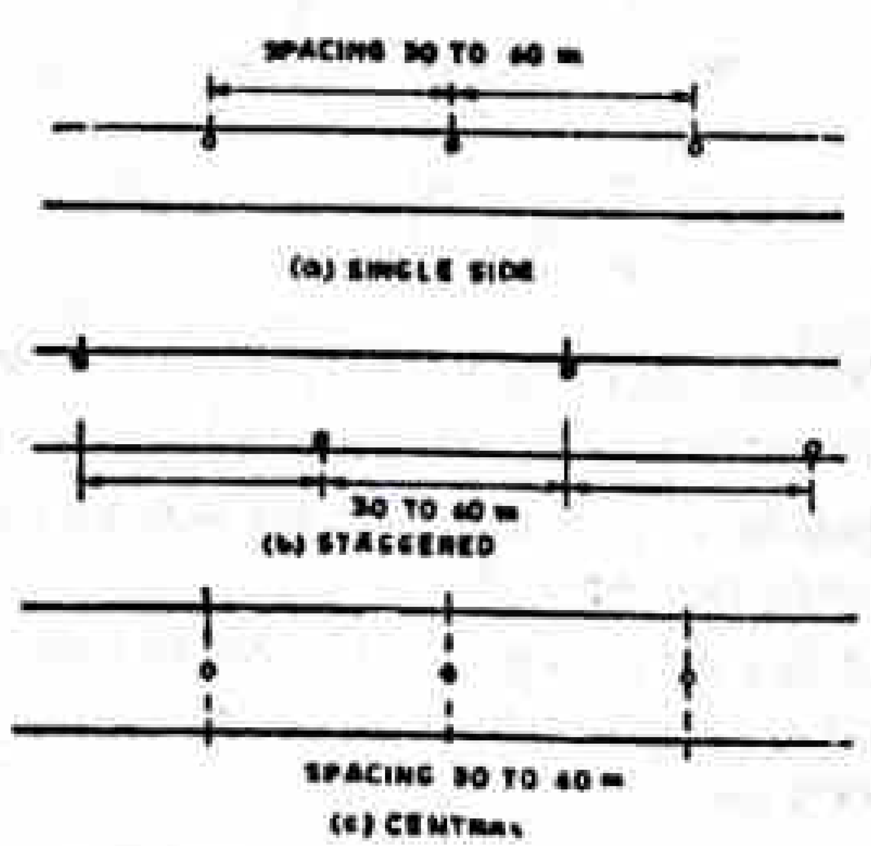


Fig. 5.43 Lighting Layouts

Special care should be taken while locating the lights on curves. Lights are installed at closer spacings on curves than on straights. The lights are located on the outer side of the curves to provide better visibility. The layout of light at horizontal curves is shown in Figure. 5.44. At vertical summit curve lights should be installed at closer intervals near the summit.

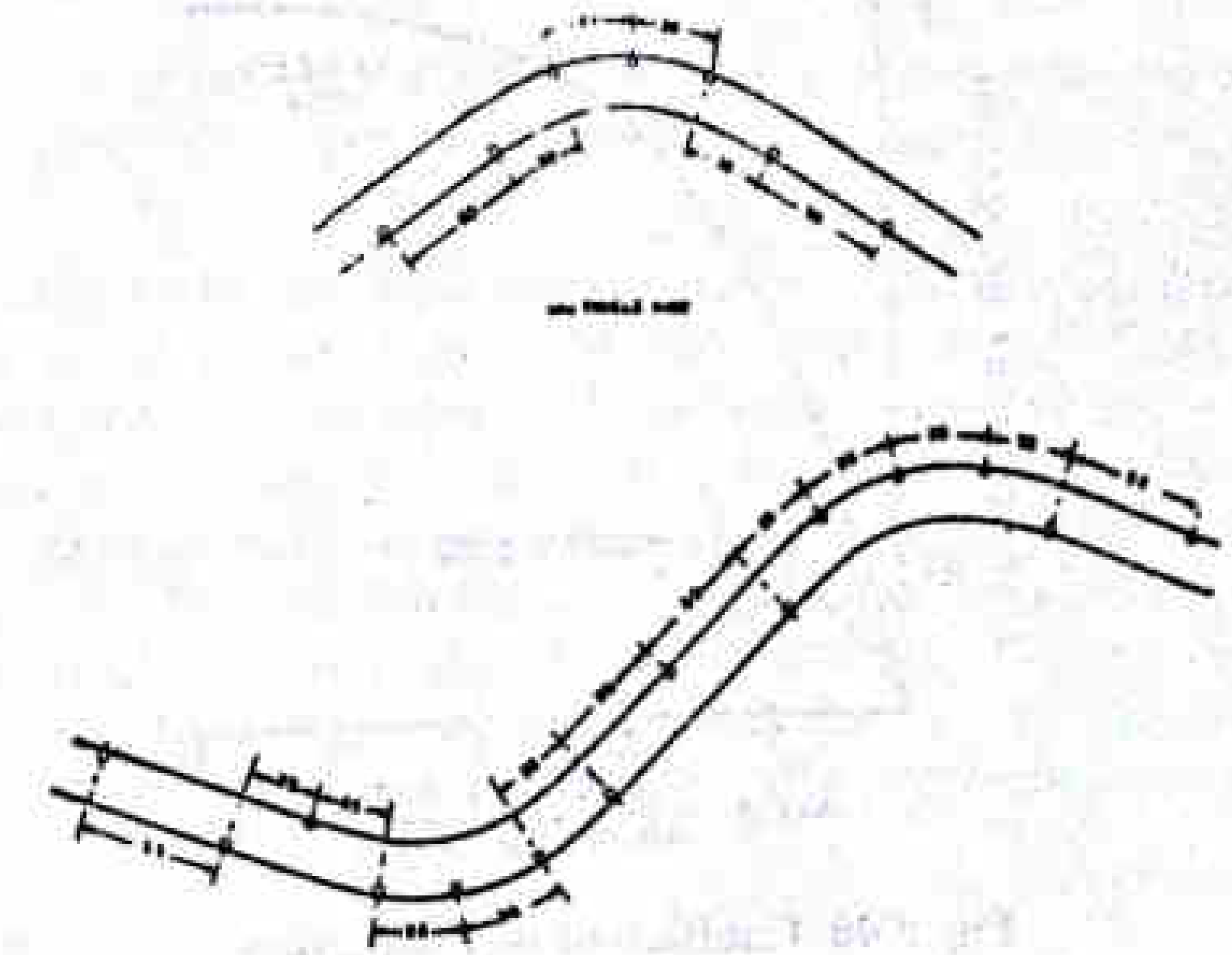


Fig. 5.44 Lighting Layout on Horizontal Curves

At intersections, due to potential conflicts of vehicular and pedestrian traffic, more illumination is required. For simple intersections, in urban area, the illumination should be atleast equal to the sum of illumination values for two roads which form the intersection. See Figure 5.45. A detailed traffic volume and flow study should be made in the cases of compound intersections before deciding the layout of lights. The lighting unit should be located near the pedestrian crossing, channelizing islands and signs. The lighting layout for traffic rotaries has been shown in Fig. 5.46.

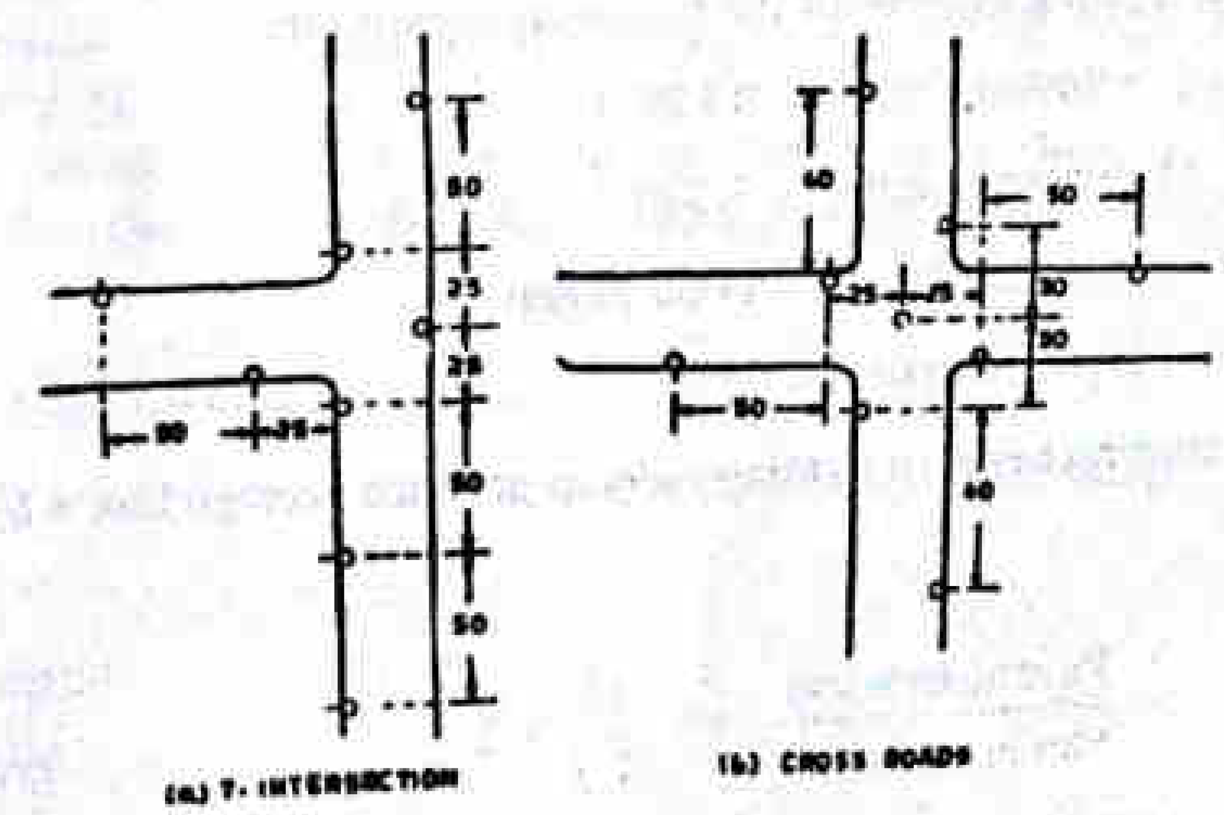


Fig. 5.45 Lighting Layout for Intersections

Design of highway lighting systems

For various types of luminaire distribution, the utilization coefficient charts are available for determination of average lux of intensity over the roadway surface where lamp lumen, mounting height, width of paved area and spacing between lighting poles are known. The typical utilization coefficient chart is given in Fig. 5.46.

The following relationship is used for computations :

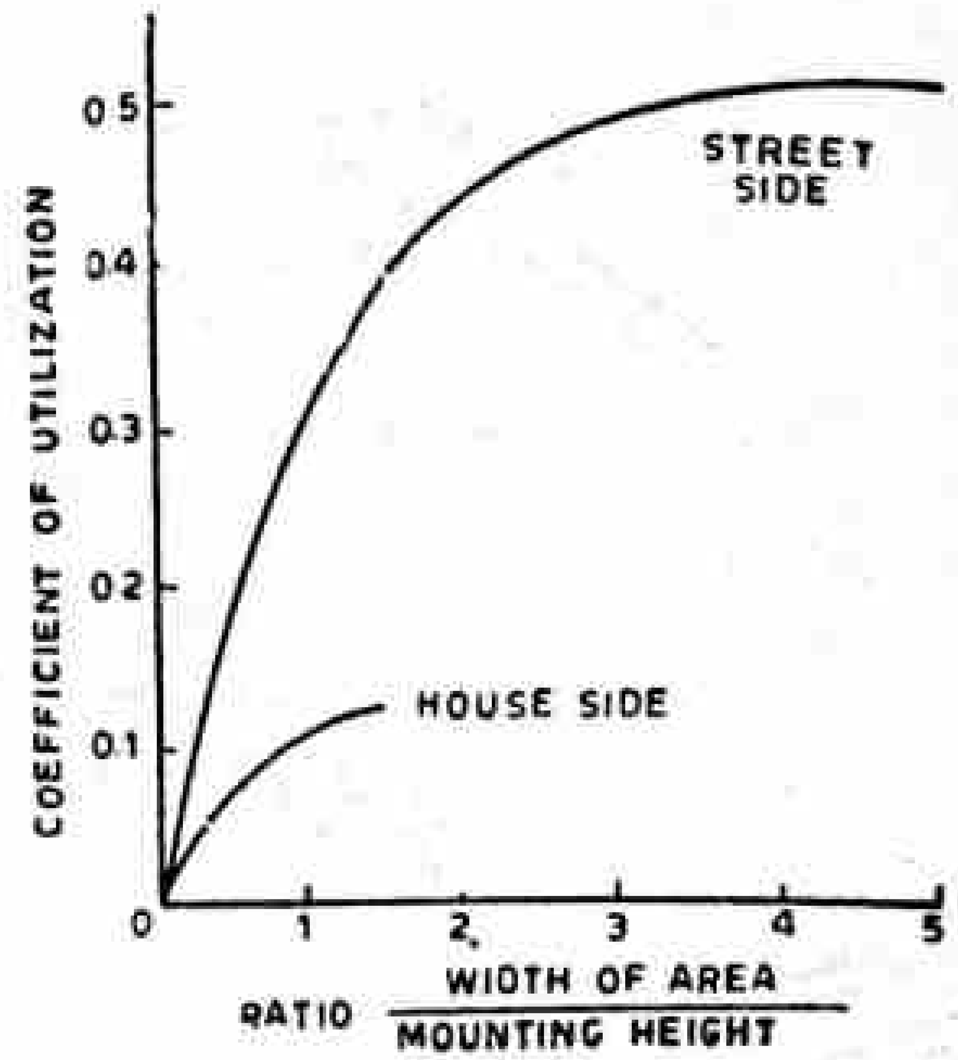


Fig. 5.46 Coefficient of Utilization

$$\text{Spacings} = \frac{\text{Lamp lumen} \times \text{Coefficient of utilization} \times \text{Maintenance factor}}{\text{Average Lux} \times \text{Width of road}}$$

The coefficient of utilization is obtained from the appropriate chart, as in Fig. 5.46. The maintenance factor takes into account the decrease in efficiency of lamp with age and an average value of about 80% may be assumed.

Example 5.16

Design a street lighting system for the following conditions

Street width – 15 m

Mounting height – 7.5 m

Lamp size – 6000 lumen

Luminaire type – II

Calculate the spacing between lighting units to produce average Lux = 6.0

Solution

The ratio $\frac{\text{Pavement width}}{\text{Mounting height}} = \frac{15}{7.5} = 2$

From Fig. 5.46, coefficient of utilization = 0.44

Assume a maintenance factor = 0.8

$$\text{Spacing} = \frac{\text{Lamp lumen} \times \text{Coefficient of utilization} \times \text{Maintenance factor}}{\text{Average Lux} \times \text{Width of road}}$$

$$= \left[\frac{6000 \times 0.44}{6 \times 15} \times 0.8\right] = 23.2$$

5.7 TRAFFIC AND TRANSPORTATION PLANNING

5.7.1 Traffic Planning

Rising trends in growth of population and traffic around cities and the steady growth of national productivity create a continuing demand for improvements in highway facilities.

The problem of traffic accidents and congestion in urban roads is being viewed with grave concern in the recent years. The main causes for this problem are improper planning of road net-work and other roadway facilities and poor traffic planning. Hence traffic functions now occupy a good position in Corporation and Municipalities. The functions and duties of traffic engineering units were initially limited to traffic surveys and control devices. But now this branch of engineering has developed considerably and include many other activities like design, regulation, planning and administrative functions. In municipal organizations a full fledged traffic engineering unit can be entrusted to look after public safety. Such a traffic engineering unit may have several division such as :

(a) Field studies
(b) Accident analysis
(c) Traffic control devices
(d) Design and planning
(e) Special investigations
(f) Economic analysis and decision theory in engineering design, and
(g) Administration

Traffic engineering units should have a proper place in highway departments or Public Works Department of the States. The financing for traffic engineering activities is another problem. Obviously, the travelling public is more concerned about their safe and quick movements and hence a provision can be made to divert part of the income obtained from the road users in the form of taxes, parking charges, tolls etc. towards these activities.

5.7.2 Urban Transportation Planning Process

The transportation planning process is developed in a series of stages :

(i) Inventories
(ii) Trip generation
(iii) Trip distribution
(iv) Model split
(v) Traffic assignment
(vi) Plan preparation and evaluation

Inventories

Information related to land use, economic activity, population, travel characteristics and transportation facilities are collected through a series of surveys. For this purpose the metropolitan area under study is sub-divided into a number of smaller zones as shown in Fig. 5.7. The following rules are normally followed for zoning :

(i) Zones should be homogeneous in land use.

(ii) Zones should be of homogeneous traffic generating characteristics.

(iii) Zones should conform to enumeration districts, natural and physical barriers.

(iv) Zones should not be large enough to produce errors resulting from the assumption that all activities occur at zonal centroid.

(v) Zones should preferably have a geometrical shape for easy determination of centroids.

Detailed surveys are than organised to assess the existing activity levels and transportation facilities. Normally Home interviews surveys, Population data, Home hold trends. Socio-economic characteristics of the population, Land use and economic activities, Traffic volume census; Travel time studies and a Physical inventory of highway net work are carried out. Some of the surveys are explained in Article 5.2.3.

The information collected are analysed with respect to past trends and future expectations forming the basis for further travel demand analysis.

Trip Generation

This is the first stage of the travel demand forecasting process. Trip generation concerns with the estimation of number of trips produced in or attracted to a given zone. The *Trip* is defined as the "One-way movement having single purpose and mode of travel between a point of origin and a point of destination". Two popular methods of trip generation estimation are

Multiple Regression Analysis

Category Analysis

In Multiple Regression Analysis a functional relationship is expressed between the trips per zone and various socioeconomic activity levels in that zone. It is usually a linear model of the form :

$$y = b_0 + b_1 x_1 + \dots\dots\dots + b_k x_k \tag{5.28}$$

where y = dependent variable-trips produced or attracted in a zone.

$x_1, x_2, \dots\dots x_k$ = independent variables that cause generation of trips.

$b_0, b_1, \dots\dots b_k$ = regression coefficients which are to be calibrated from the base year data obtained in inventory studies.

As an example in one of the Indian cities, the following relationship was established :

$$y = 1197.32 + 0.0957\, x_2 \tag{5.29}$$

where x_1 = number of workers in the zone

x_2 = number of vehicles in the zone

and y = number of trips produced for work purposes in a zone

On the assumption that this model remains stable over time, the future number of trips likely to be generated can be predicted by substituting the future estimates of the $x_1, x_2, \dots\dots x_k$, the independent variables in the equation.

In the category analysis, the household trip making is considered rather than zonal trip making. The procedure is to divide the households into a set of categories and to determine the base year trip rates to each category. In England 108 different categories are normally used based on 6 income classes, 3 car ownership levels, and 6 house hold structures. To predict the future trip generation, the expected number of households in such category at the design year are to be multiplied by the corresponding trip rate and summed up over the zone.

Trip Distribution

Trip distribution is the stage where the trips generated and attracted from each zone are distributed to any other zone. The most important method for this procedure is the *Gravity Model*. This model is based on the principle that the trips between any two zones i and j are directly proportional to the number of trips generated in the zone i, the number of trips attracted to zone j, and are inversely proportional to some function of distance or separation between the zones.

The model is as follows :

$$T_{ij} = \frac{G_i\, A_j\, F_{ij}}{\sum_{j=1}^{n} A_i\, F_{ij}} \tag{5.30}$$

Here T_{ij} = number of trips from zone i to zone j

G_i = Trips generated in zone i

A_j = Trips attracted to zone j

F_{ij} = Empirically derived 'friction factor' calculated on area wise basis

n = number of zones in the urban area.

Existing data is used initially to calibrate the mode parameters F_{ij} through a computer program. Assuming these parameters to be same at a future date, the future trip interchanges are computed by substituting the future trip generated values in the model.

Model Split

The proportion of total trips between any two zones that can be shared between the private vehicles and the public transportation system is determined in this stage. The models so far developed have been designed to determine this proportion between car and bus modes. Several approaches are available to solve this problem. Typically the times and costs of travel by car mode and bus mode are assessed between two O-D points. With the help of diversion curves the number of bus trips likely to be made within the O-D pair is then determined.

Traffic Assignment

The next stage in the transportation planning is the assignment of various trips between any two O-D pairs on different highway routes. A typical method is known as "All-or-Nothing" procedure. In this the shortest route between the given O-D pair is identified. All the trips between these two zones are then assigned to this path. Similarly the trips between any two O-D pairs are assigned to respective shortest routes. A number of other complex methods are also available and give more realistic traffic assignments.

Plan Preparation and Evaluation

The steps described above will enable the various land use strategies and travel demands to be explored. In very general terms, a set of objectives are specified and various land use and transportation plans are developed. The benefits and losses likely to be caused by any alternative and the net returns with the estimated investment are then compared. An economic criterion is used to choose the best alternative by a critical analysis.

Example 5.17

The following information was obtained from a transportation survey of a town :

Traffic zone number	Population in the zone (thousands)	Total trips generated (in hundreds)
1	26	12
2	28	11
3	31	17
4	33	15
5	22	12
6	30	15
7	20	9
8	25	13

Develop a linear regression model for estimating the trips generated from a zone. If the population in a particular zone increases to 40,000, predict the expected trip generation from that zone.

Solution

(a) In this case there is one independent variable population and the problem is to develop a linear equation of the type.

$$y = b_0 + b_1 x_1$$

Here y = Total number of trips in hundreds per zone, being the dependent variable.

x_1 = Population of a zone in thousands, being the independent variable.

b_0 = Regression constant or intercept term.

b_1 = Regression coefficient

The equation is calibrated for b_0 and b_1 by the following formulae :

$$b_1 = \frac{n\Sigma xy - \Sigma x \Sigma y}{n\Sigma x^2 - (\Sigma x)^2} \quad (5.31)$$

Also the correlation coefficient r, which tells about the goodness of fit is obtained by :

$$r = b_1\left[\left(\frac{n\Sigma x^2 - (\Sigma x)^2}{n\Sigma y^2 - (\Sigma y)^2}\right)\right]^{1/2} \quad (5.32)$$

Here n = total number of observations

Σ = Summation of the variable over all the observations

It is convenient to prepare a tabular form for computations :

Zone	x	y	xy	x^2	Y^2
1	26	12	312	676	144
2	28	11	308	784	121
3	31	17	527	961	289
4	33	15	495	1089	225
5	22	12	264	484	144
6	30	15	450	900	225
7	20	9	180	400	81
8	25	13	325	625	169
n = 8	$\Sigma x = 215$	$\Sigma y = 104$	$\Sigma xy = 2861$	$\Sigma x^2 = 5919$	$\Sigma y^2 = 1398$

Substituting these values in the formulae given earlier

$$b_1 = \frac{8 \times 2861 - 215 \times 104}{8 \times 5919 - (215)^2} = 0.469$$

$$b_0 = (104 - 0.469 \times 215)/8 = 0.396$$

Therefore, the trip generation model is

$$y = 0.396 + 0.469\ x$$

The correlation coefficient for this model is

$$r = 0.469\left[\left(\frac{8 \times 5919 - (215)^2}{8 \times 1398 - (104)^2}\right)\right]^{1/2} = 0.82$$

The linear regression model is given by :

$$y = 0.396 + 0.469\ x,\ r = 0.82$$

(b) The future population of a zone = 40,000

$$x = 40 \text{ for use in model}$$

The total trips generated, $y = 0.396 + 0.469 \times 40 = 19.16$ in hundreds = 1916

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