Special Publication 41

GUIDELINES FOR THE DESIGN OF AT-GRADE INTERSECTIONS IN RURAL & URBAN AREAS

THE INDIAN ROADS CONGRESS

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GUIDELINES FOR THE DESIGN OF AT-GRADE INTERSECTIONS IN RURAL & URBAN AREAS

1. INTRODUCTION

1.1 The question of preparation of Guidelines for the Design of At-Grade Intersections has been under the consideration of the Traffic Engg. Committee for some time past. The intersections are important elements of road and at-grade intersections are very common on Indian roads. There has been a long felt need of framing some guidelines for the design of at-grade intersections in rural and urban areas which are readily usable by the road authorities and practising engineers. The initial draft was prepared by Shri J.B. Mathur and later on given final shape by the Working Group constituted for the purpose consisting Dr. A.C. Sarna, Dr. A.K. Gupta, S/Shri D. Sanyal, J.B. Mathur, Vishwanathan and M.K. Bhalla based on the comments received from the members of Traffic Engg. Committee. The draft thus finalised was discussed in the meeting of Traffic Engg. Committee (personnel given below) held on 2nd December 1991 and approved with slight modifications.

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1.1.1. The draft guidelines were considered by the Highways Specifications & Standards Committee in its meeting held on 1.9.92 and approved with some modifications. The modified draft was subsequently approved by the Executive Committee and Council in their meetings held on 11.11.92 and 28.11.92 respectively. The draft was finally modified by Shri A.P. Bahadur and the IRC Sectt. in consultation with the Convenor, Highways Specifications & Standards Committee as authorised by the Council for printing as one of the IRC Publication.

1.2. Scope

These guidelines are intended to assist those who are required to design or improve at-grade intersections in rural and urban areas. It takes into account the mixed and heterogeneous traffic conditions prevailing in India. As the guidelines encompass wide range of conditions prevailing in different parts of rural, urban and hilly areas, it is necessary that the users of these guidelines apply their knowledge of local conditions in interpreting and arriving at a correct solution.

These guidelines cover at-grade intersections but not the design of interchanges which is covered by IRC : 92-1985 "Guidelines for the Design of Interchanges in Urban Areas". Other standards which have associated application include IRC : 93-1985 "Guidelines on Design and Installation of Road Traffic Signals" and IRC : 65-1976 "Recommended Practice for Traffic Rotaries". Contents of these publications are repeated here only to the extent relevant.

1.3. Factors Covering Design

1.3.1. Road intersections are critical element of a road section. They are normally a major bottleneck to smooth flow of traffic and a major accident spot. The general principles of design in both rural and urban areas are the same. The basic difference lies in the design speeds, restriction on available land, sight distance available and the presence of larger volume of pedestrians and cyclists in urban areas.

1.3.2. Design of a safe intersection depends on many factors. The major factors can be classified as under :

A. Human Factors

1. Driving habits,
2. Ability to make decisions,
3. Driver expectancy,
4. Decision and reaction time,
5. Conformance to natural paths of movement,
6. Pedestrian use and habits.

B. Traffic considerations

1. Design and actual capacities,
2. Design hour turning movements,
3. Size and operating characteristics of vehicles,

4. Types of movement (diverging, merging weaving, and crossing),
5. Vehicle speeds,
6. Transit involvement,
7. Accident experience,
8. Traffic Mix i.e. proportion of heavy and light vehicles, slow moving vehicles, cyclists etc.

C. Road and Environmental considerations

1. Character and use of abutting property,
2. Vertical and horizontal alignment at the intersection,
3. Sight distance,
4. Angle of the intersection,
5. Conflict area,
6. Speed-change lanes,
7. Geometric features,
8. Traffic control devices,
9. Lighting equipment,
10. Safety features,
11. Environmental features,
12. Need for future upgrading of the at-grade intersection to a grade separated intersection.

D. Economic factors

1. Cost of Improvements,
2. Effects of controlling or limiting right-of-way on abutting residential or commercial properties where channelisation restricts or prohibits vehicular movements.

1.4. Intersection Types and Choice

1.4.1. Generally intersections can be classified into three categories depending on the traffic conditions. These are :

(1) Uncontrolled Intersections at-grade : These are the intersections between any two roads with relatively lower volume of traffic and traffic of neither road has precedence over the other.

(2) Intersection with Priority Control : There is theoretically no delay occurring on the major road and vehicles on the minor road are controlled by “GIVE - WAY” or “STOP” sign.

(3) Time separated intersection/Signalised Intersections at-Grade : The detailed warrants for signalised intersection are laid down in IRC : 93-1985. A signalised intersection besides other warrants, is justified if the major street has a traffic volume of 650 to 800 vehicles per hour (both directions) and minor street has 200 to 250 vehicles per hour in one direction only.

(4) Space Separated Intersection/Grade Separated Intersections : The detailed warrants for interchange or grade separated Intersections are given in IRC : 92 - 1985. According to these, a grade-separated intersections, besides other warrants, is justified when the total traffic of all the arms of the intersection is in excess of 10,000 PCU’s per hour.

1.4.2. A graphical relationship has been developed in UK with the help of which, a selection can be made on type of intersection required, based on traffic flows, in urban conditions. This is shown in Fig 1.1. The regions with dotted lines between priority, round about and grade separation are the areas where the selection between the two shall also be governed by other considerations, such as the availability of space & costs etc.

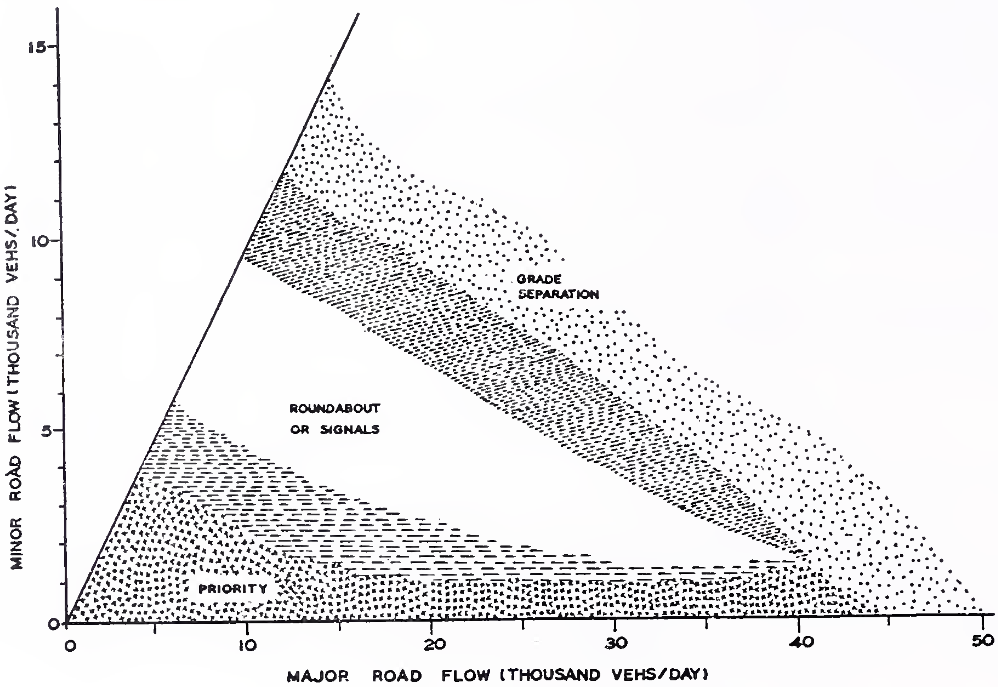


Fig. 1.1. Intersection Selection based on Traffic Flow Combination (U.K. Practice)

1.4.3. Road Intersections occur in multiplicity of shapes. They can, however, be divided into seven basic forms - T, Y, Scissor, Cross, Staggered, Staggered and skewed, and multiway. The various shapes of at-grade intersections are shown in Fig. 1.2. The relation of a particular shape is governed by the configurations and geometrics of the intersecting arms.

2. BASIC DESIGN PRINCIPLES

In the design of an intersection the primary considerations are safety, smooth and efficient flow of traffic. To achieve this, the following basic principles must be followed.

2.1. Uniformity and Simplicity

Intersections must be designed and operated for simplicity and uniformity. The design must keep the capabilities and limitation of drivers, pedestrians and vehicles using

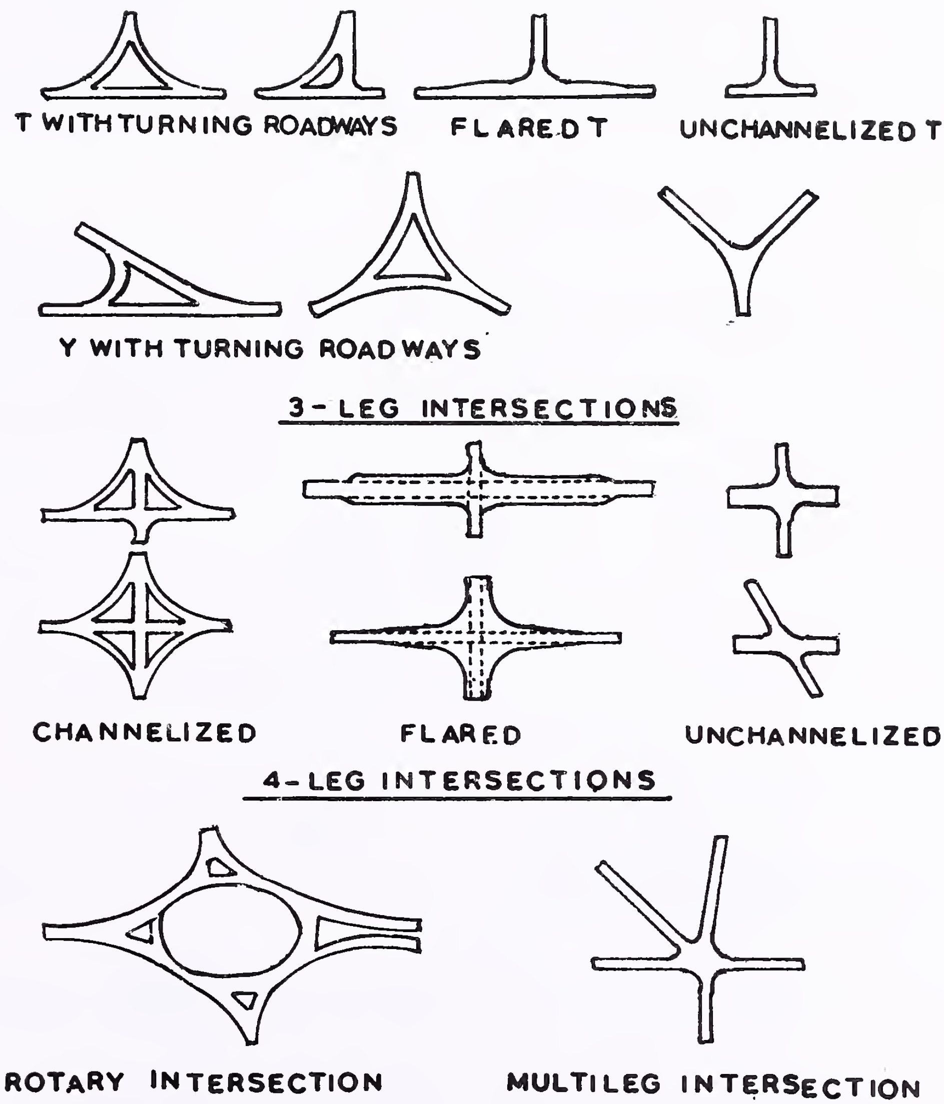


Fig. 1.2. General Types of At-Grade Intersections

intersection. It should be based on a knowledge of what a driver will do rather than what he should do. Further all the traffic information on road signs and markings should be considered in the design stage, prior to taking up construction work. All the interesection movements should be obvious to the drivers, even if he is a stranger to the area. Complex design which require complicated decision-making by drivers should be avoided. There should be no confusion and the path to be taken by the drivers should be obvious. Undesirable short cuts should be blocked. Further, on an average trip route, all the intersections should have uniform design standards so that even a newcomer to the area anticipates what to expect at an intersection. Some of the major design elements in which uniformity is required are design speed, intersection curves, vehicle turning paths, super-elevations, level shoulder width, speed change lane lengths, channelisation, types of curves and type of signs and markings.

2.2. Minimise Conflict Points

2.2.1. Any location having merging, diverging or crossing manoeuvres of two vehicles is a potential conflict point. Fig. 2.1 shows the potential conflict points for different types of intersections. The main objective of the intersection design is to minimise the number and severity of potential conflicts between cars, buses, trucks, bicycles and pedestrians and whenever possible, these should be separated. This can be done by :

(i) Space separation : by access control islands through channelising

(ii) Time separation : by traffic signals on waiting lanes.

Some of the common methods used to reduce conflict points are :

(a) Convert a 4-armed intersection having 32 conflict points to a roundabout having only 12 conflict points. Round-about treatment may not, however, be warranted at most of rural locations except those close to the urban areas.

(b) Signalise intersection. As Fig. 2.1 shows introduction of a two-phase signal reduces the conflict points at 4 armed intersection from 32 to 16. If more phases are introduced and separate lanes provided for turning traffic, conflict points can be virturally eliminated. (Provision of signals may however, be justified only at a few rural locations carrying heavy traffic). Research abroad has shown that signals increase accidents at simple intersections with low volumes but reduce them at complex and/or high volume intersections.

(c) Channelising the directional traffic by selective use of channelising islands and medians. Some of these techniques are shown in Fig. 2.2. Fig. 2.3 shows how the conflict points can be reduced on a 3-armed intersection by introducing combinations of channelising islands.

(d) Changing priority of crossing by introducing the GIVE WAY or STOP signs for traffic entering the junctions from minor road. By this, traffic causing the conflict is restrained.

(e) Staggering a 4-armed junctions by flexing the two opposing arms of the side road to create two T-junctions. When staggering is employed, it should be ensured that minimum distance between two junctions is 45 m and desirably right-left staggers are created (Fig. 2.4).

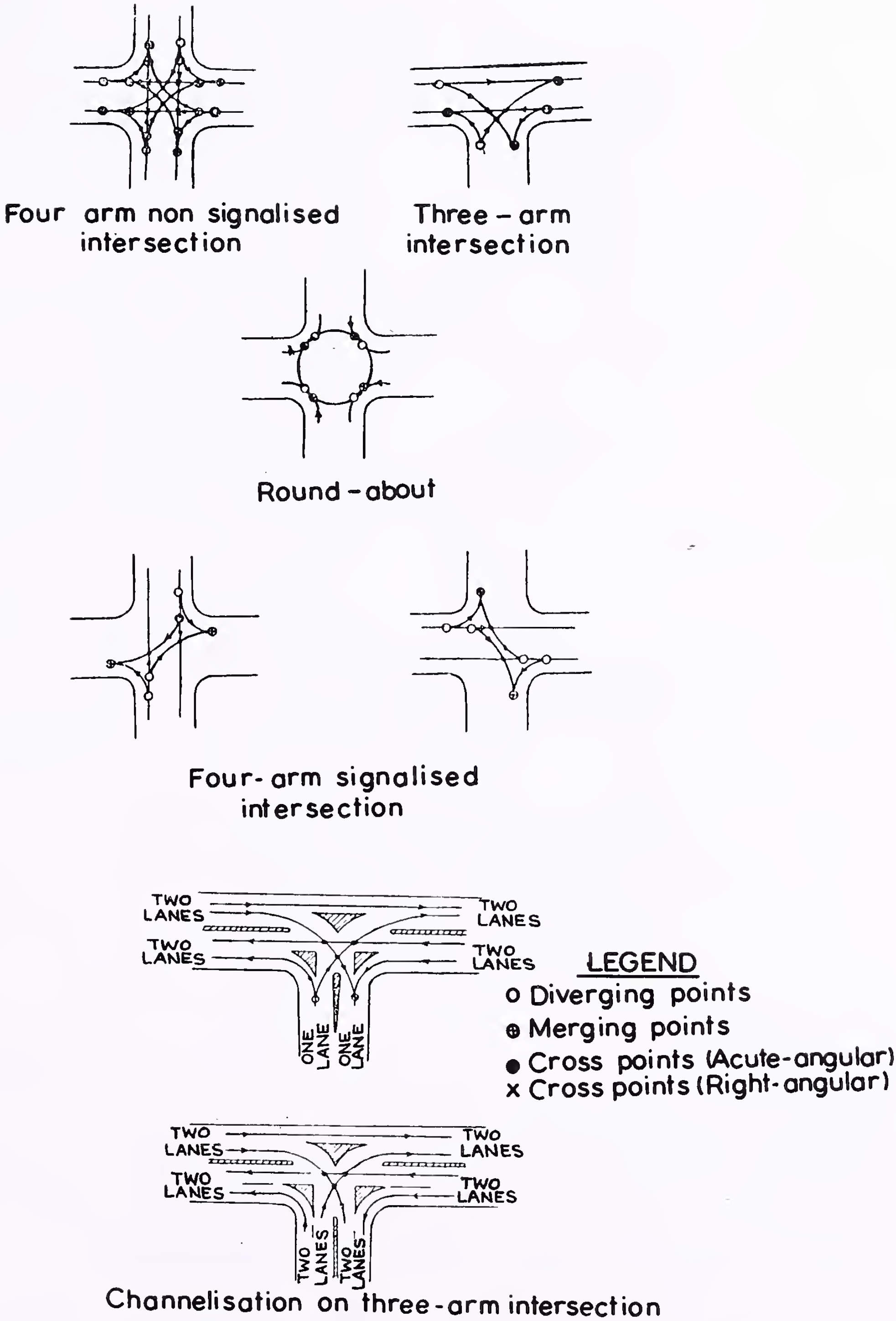


Fig. 2.1. Potential Conflict Points at Different Types of Intersections

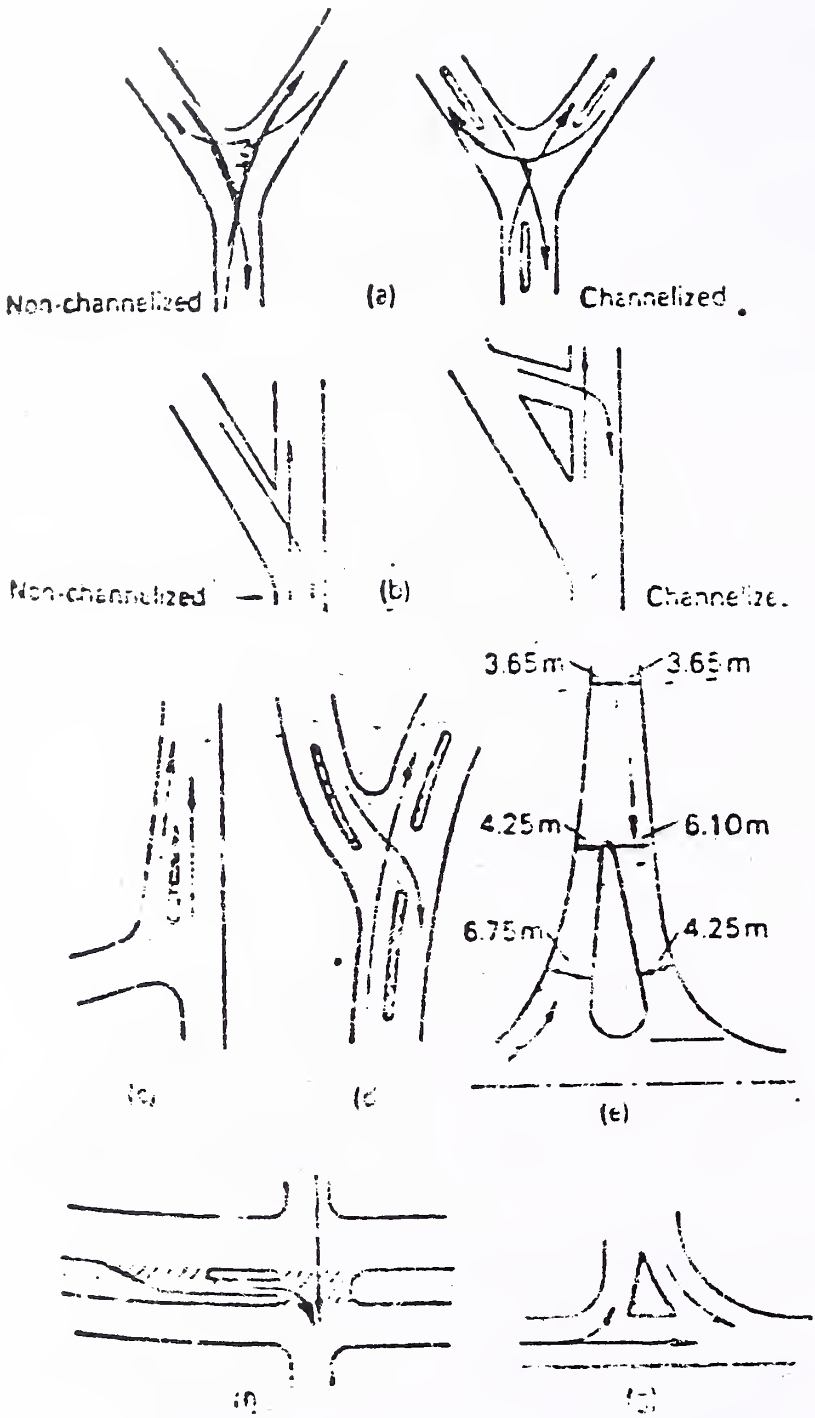


Fig. 2.2. Channelisation Techniques Illustrating Basic Intersection Design Principles

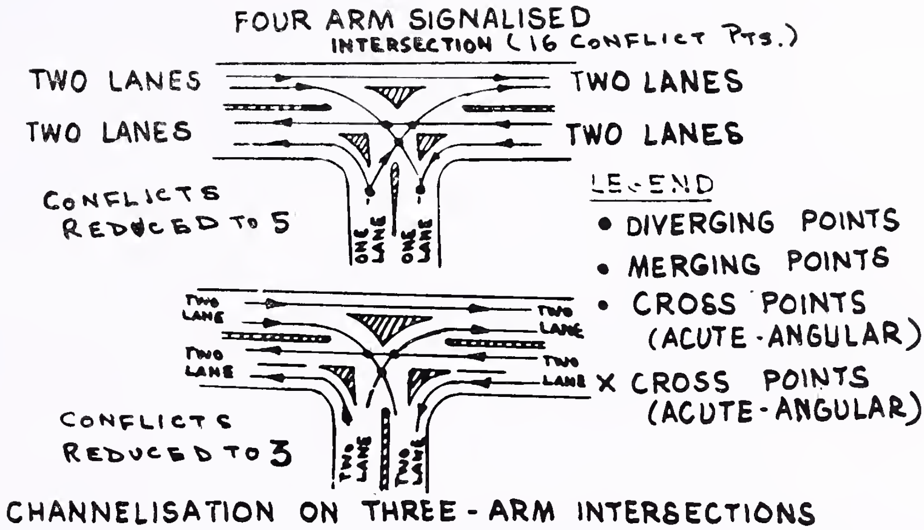


Fig 2.3. Potential Conflict Points at Different Types of Intersections

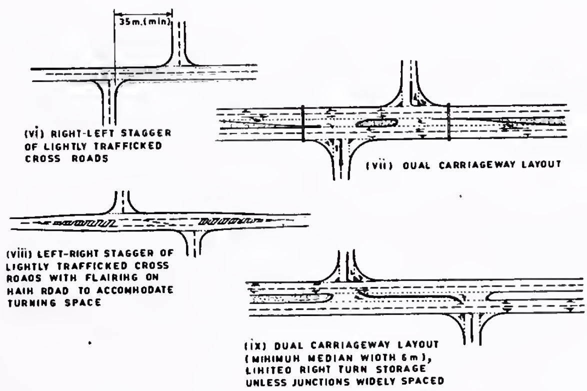


Fig. 2.4. Staggering of Intersections

2.2.2 A study of conflict points and accident records by classifying accidents according to the types of conflicts would greatly help in adopting appropriate engineering measures for intersection design. For illustration, an example of a 3-armed intersection as shown in Fig. 2.5 may be considered. The Figures (a) to (i) show the various left turning & right turning movements, and the percentage of accidents classified according to types of collision. Following measures can be considered for improving safety :

(i) The accident situations in Figs 2.5 (c), (d) and (h) involving right turning vehicles in high percentage of accidents can be prevented by controlling traffic movements either manually or by traffic signals.

(ii) Provision of acceleration lane on the major road for the left turning traffic flow from the minor road could prevent traffic situation, Fig. 2.5(f).

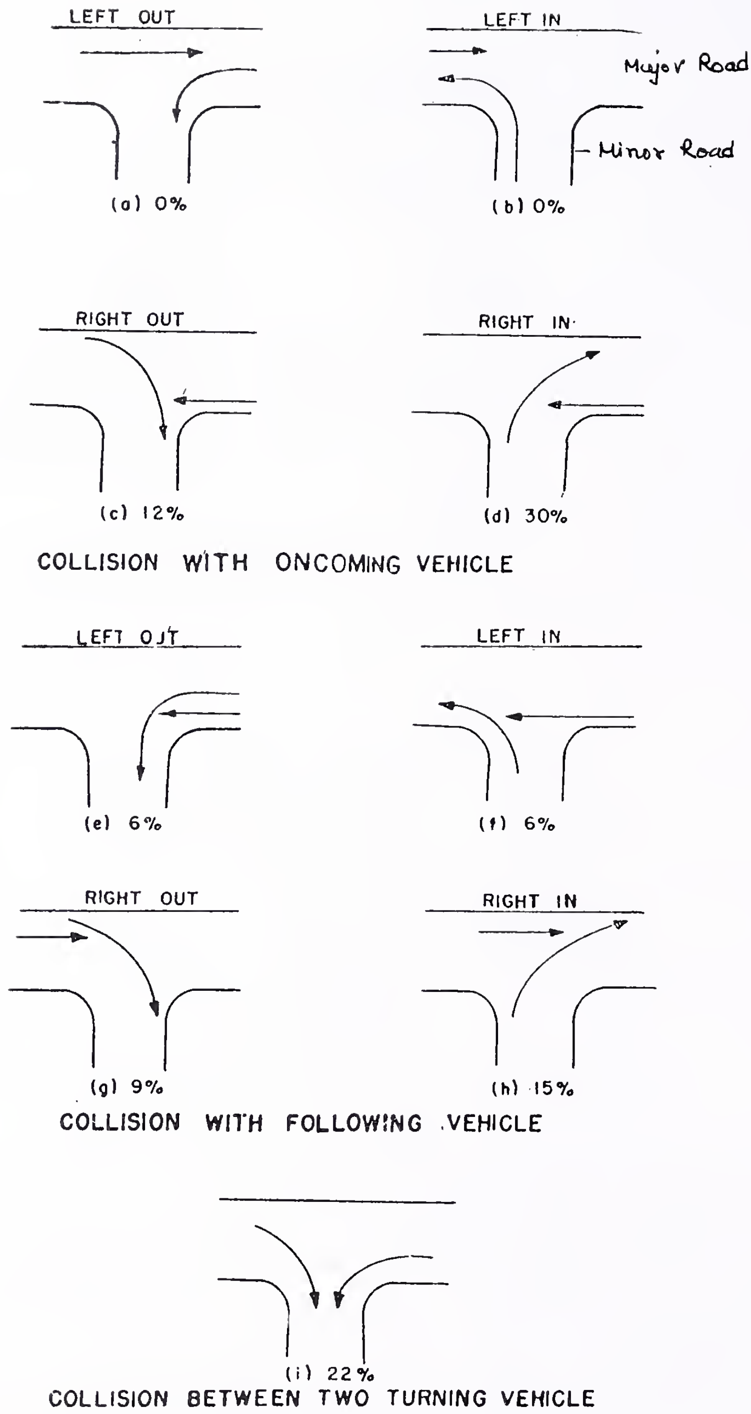


Fig. 2.5. Analysis of Accident Types at Three Arm Intersections

(iii) A separate right turning lane on the major road could minimise or prevent rear end collision at Fig. 2.5 (g).

(iv) Channelising islands at the minor road could be useful in situation at Fig. 2.5 (i).

(v) Deceleration lanes for left turning traffic from major road would also ensure better safety for situations at Fig. 2.5(e).

2.3. Safety

2.3.1. The safety of a particular design can best be assessed by studying the frequency with which types of accidents occur at particular type of intersection and its correlation with volume and type of traffic. Refer to para 3.1 (vi) for details. It is, therefore, necessary that a systematic record be maintained of all accidents at intersections in Road Accidents Forms suggested in IRC : 53-1982 (under-revision).

2.3.2. Prioritisation of intersection improvements can be done using the relationship of accident frequency with traffic volumes. A simple equation developed in U.K. is in the form

$$C = \frac{A}{\sqrt{Qq}}$$

Where A is the number of accidents in a year, Q and q are traffic volumes on the main and side roads in thousands of vehicles per day. Intersections with higher C values are considered for priority treatment.

2.3.3. Some of the measures which could enhance safety at intersections are :

(i) By eliminating highly trafficked side road connections, in rural sections upto 30 per cent reductions in accidents can be made.

(ii) By converting lightly trafficked cross-road into properly designed staggered junction, 60 per cent reduction in accidents is possible.

(iii) In urban areas, control of access, street parking and development in the vicinity of intersection improves the safety considerably.

2.4. Alignments and Profile

2.4.1. In hilly and rolling terrain, site condition governs the alignment and grade of the intersections, but the safety can be considerably improved by designing the intersection with modification in alignment and grades. Some useful points are:

(i) The intersecting roads shall meet at or nearly at right angle. However, angles above 60° do not warrant realignment. Realignments of intersection may be in any of the forms shown in Fig. 2.6.

(ii) Intersection on sharp curves should be avoided because the superelevation and widening of pavement complicates the design.

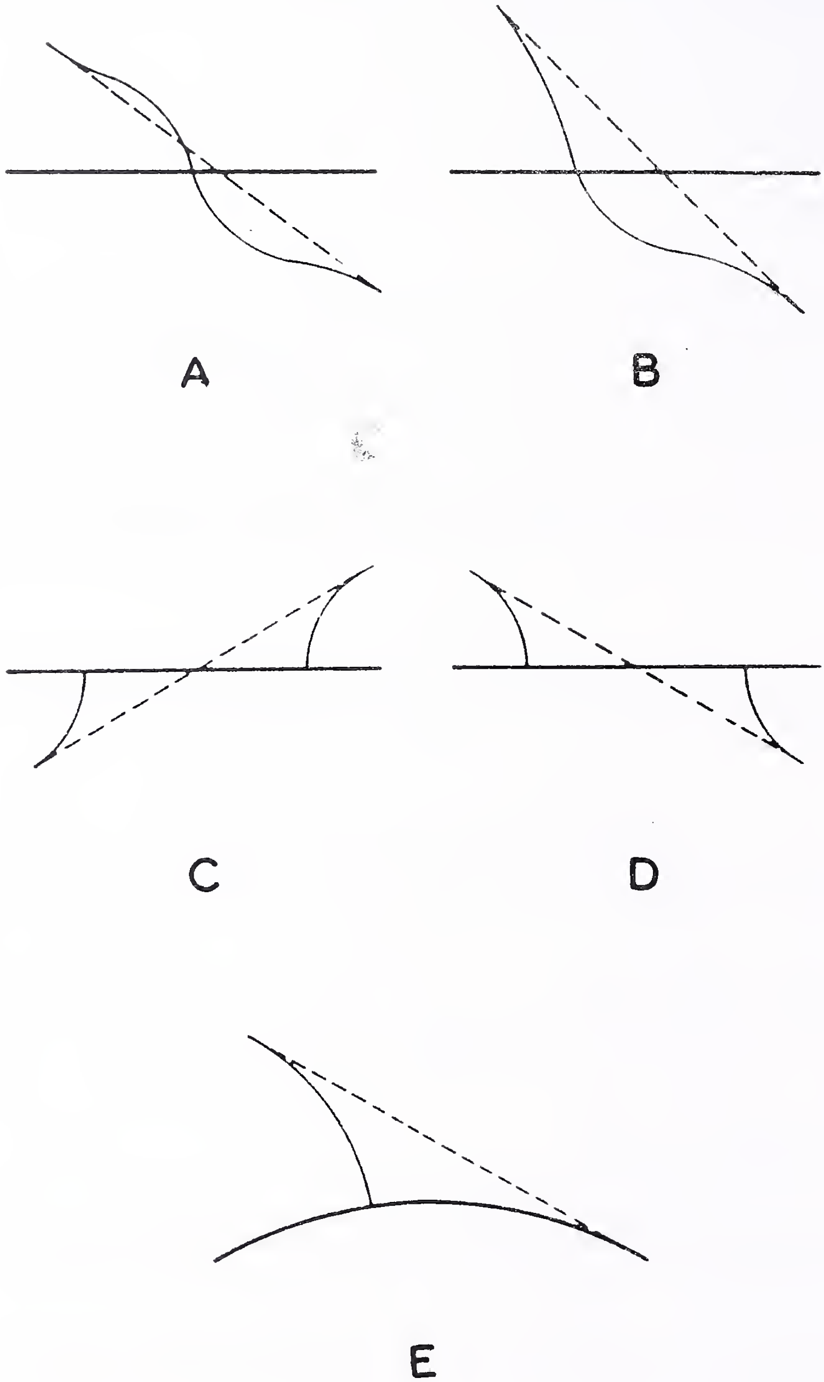


Fig. 2.6. Realignment Variation of Intersection

(iii) Combination of grade lines or substantial grade changes should be avoided at intersection. The gradient of intersecting highways should be as flat as practicable upto sections that are used for storage space.

(iv) Grades in excess of 3 per cent should, therefore, be avoided on intersections while those in excess of 6 per cent should not be allowed.

(v) Normally, the grade line of the major highway should be carried through the intersection, and that of the cross road should be adjusted to it. This concept of design would thus require transition of the crown of the minor highway to merge with the profile of the interface of major and minor roads (see Fig. 2.7).

(vi) For simple unchannelised intersections involving low speed and stop signals or signs, it may be desirable to warp the crowns of both roads into a plane at the intersection, the particular plane depending on direction of drainage and other conditions.

(vii) Changes from one cross slope to another should be gradual. Intersection of a minor road with a multilane divided highway having a narrow median and superelevated curve should be avoided whenever possible because of the difficulty in adjusting grades to provide a suitable crossing.

3. DESIGN DATA REQUIRED

3.1. In order to be able to properly design an intersection and give consideration to factors affecting design, the following essential data must be collected :

(i) An index/location plan in the scale of about 1 : 10,000 to 1 : 20,000 showing the intersection under consideration and the road/rail/river network in the area.

(ii) A base plan of the intersection site in the scale of 1 : 500. Where two or three intersections are located close together, additional base plan to a scale of 1 : 1,000 should be prepared showing all the intersections affected. It is important to maintain this scale which is being adopted as a measure of uniformity and also to ensure that sufficient length of roads and fairly detailed account of existing features are shown in a drawing sheet of manageable size. The existing roads and salient features like road land boundary, location of structures trees, service lane etc., should be shown for a length of about 200 m for each road merging at the intersections. If the terrain is not plain and/or there is too much of variation of ground level at the site, contours at 0.5 metre interval should also be marked on the base plan and additional longitudinal sections given along the centre line of intersecting roads.

(iii) The peak hour design traffic data : The peak hour design traffic data should give its compositional and the directional break-up. A sample proforma, which is to be used for the purpose of reporting the compositional and directional break up and computing the volume in PCUs for one leg of a four legged intersection, is given as Table 3.1

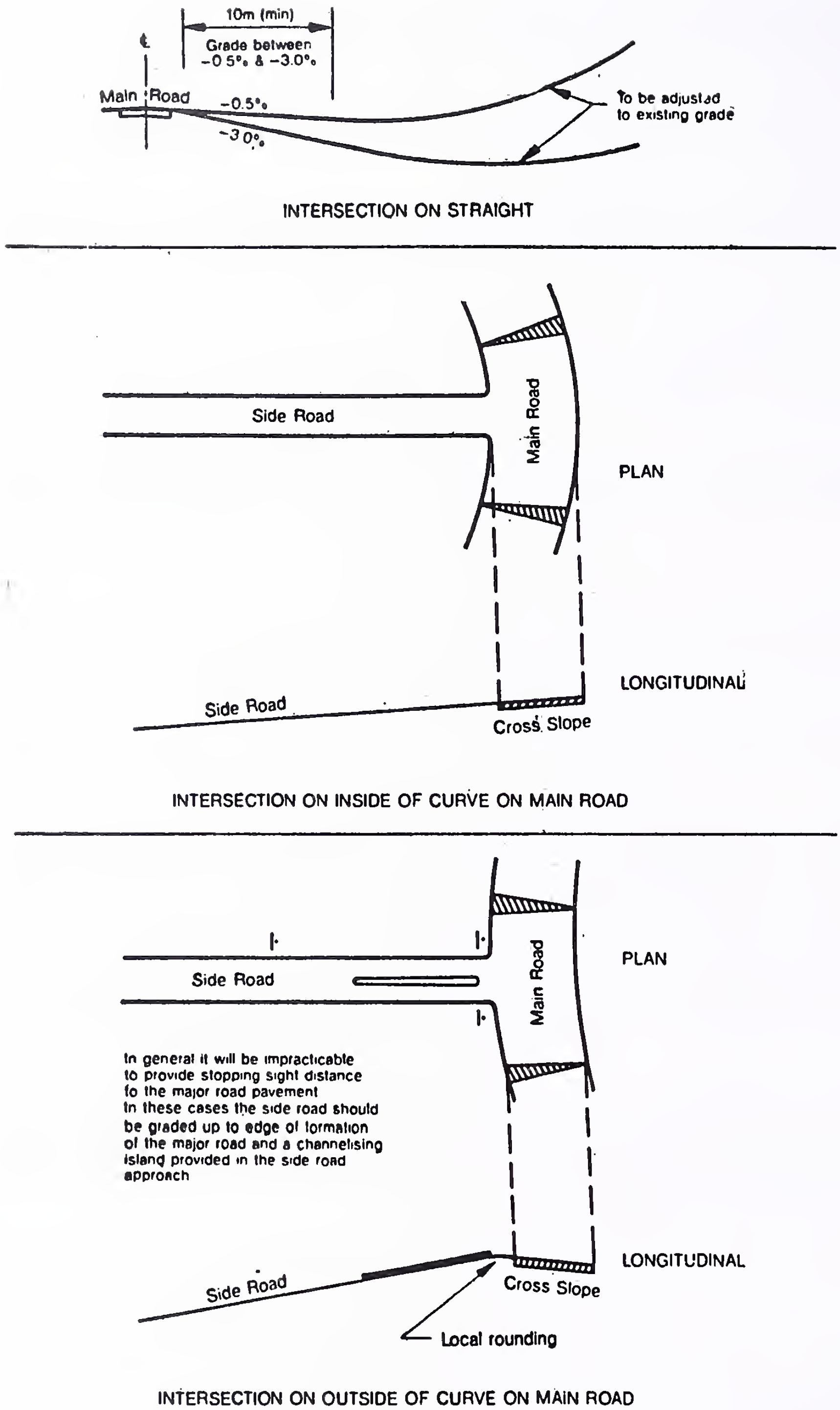


Fig. 2.7. Main Road Intersections—Approach Grading on Side Roads

Table 3.1. Intersection Design Data

Intersection Design Data **Peak Hour __________ Hrs. To ________ Hrs**
Peak Hour Design Traffic

Name & Location of Intersection ____________________________________

	From			Leg A*				
Entering	Leg B*			Leg C*		Leg D*		Remarks
Type	Nos	PCU Equi-valency	PCU	Nos	PCU Equi-valency	Nos	PCU Equi-valency	
	1	2	3=1x2	1	2	1	2	
Fast Vehicles								
1. Passenger cars, tempos auto rickshaw, tractors, pickup vans		1.00						
2. Motor Cycles, scooters		0.50						
3. Agricultrual tractor Light Commercial Vehicles		1.50						
4. Trucks, Buses,		3.00						
5. Tractor-Trailer, Truck Trailer units		4.50						
TOTAL FAST								
Slow Vehicles								
6. Cycles		0.50						
7. Cycle Rickshaws		1.50						
8. Hand Cart		3.00						
9. Horse Drawn		4.00						
10. Bullock-Carts		8.00						
TOTAL SLOW								
PEDESTRIAN Nos.								

*Specify the name of an important place or land on this LEG such as Market LEG, Temple LEG, Mathura LEG, etc.

For converting vehicles into PCUs, equivalency factors given in Table 3.1 should be used. Separate report sheets will be needed for the other legs of the intersection. The volume of the above traffic in terms of number of vehicles and in PCU should then be reflected in the diagrams shown in the Figs. 3.1 & 3.2. If the numbers of legs in the intersection are 3 or more than 4, these figures should be suitably modified.

(iv) In the urban/sub-urban areas and intersection near villages with substantial pedestrian movements, the peak hour data on persons crossing the intersecting road arms should be collected for the design of a well planned pedestrian crossing facility at the intersection.

(v) Other relevant details such as the feasibility of providing proper drainage and lighting system at the intersection and also the present and future land use in the vicinity of intersection shall be given.

(vi) Accident data at intersection should be collected as per IRC : 53-1982 in Form A-1 and data for one year should be tabulated as shown in Fig. 3.3. This should then be reduced to diagrametrical form. Study of this data on collision diagram would itself indicate the necessary engineering measures required at the intersection.

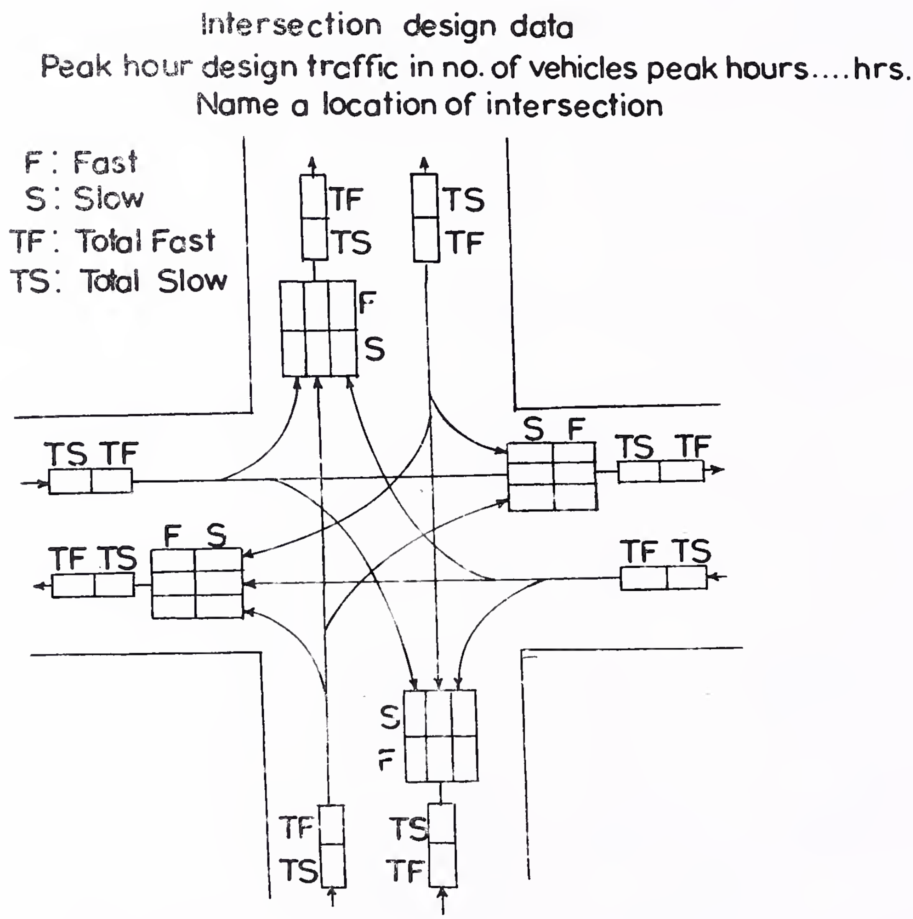


Fig. 3.1. Peak Hour Traffic Flow Diagram in Number of Vehicles

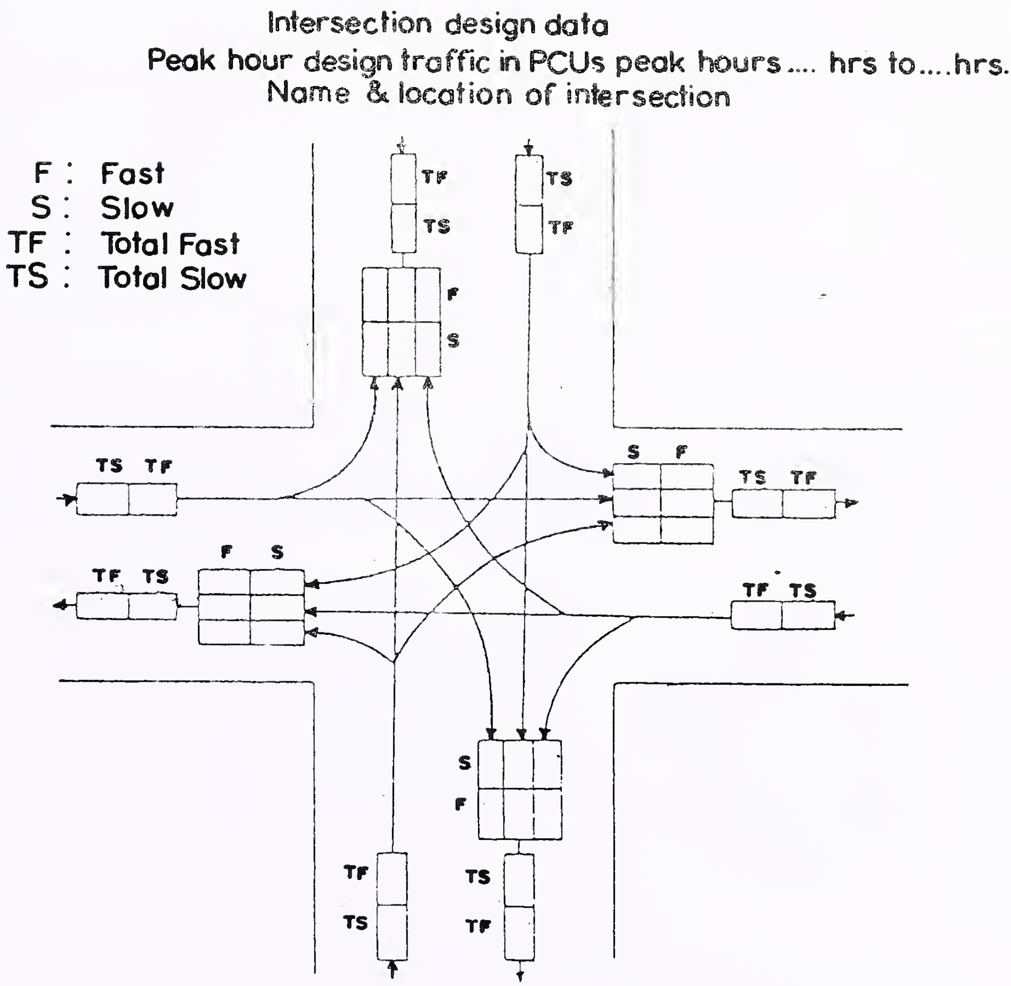
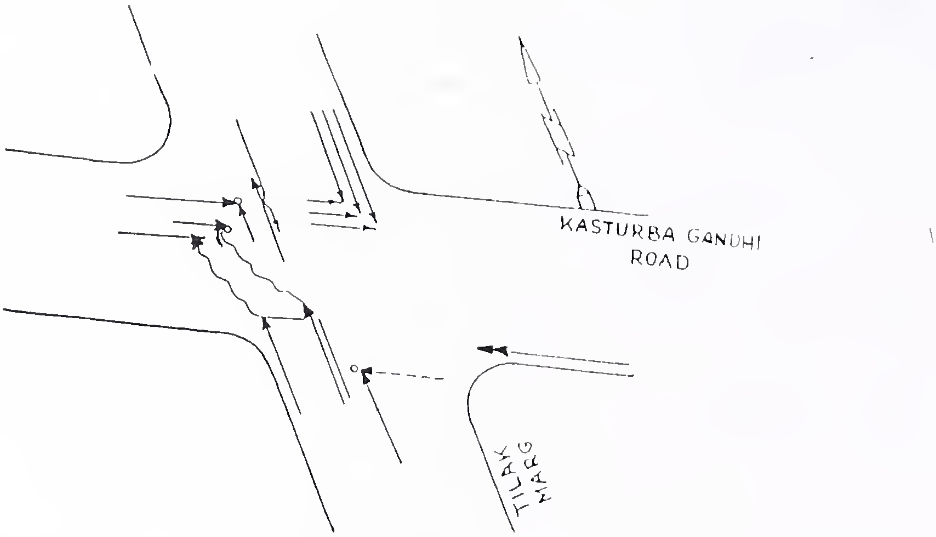


Fig. 3.2. Peak Hour Traffic Flow Diagram in PCUs

3.2. The specific form of intersection design depends on the physical conditions of the site such as topography, available right-of-way, land use and developments along the intersecting roads expected volumes of through and turning traffic including their composition, maintenance of the intersecting highway etc. The above mentioned data is meant to provide this.



LEGEND

PATH OF MOVING MOTOR VEHICLE ⟶
PEDESTRIAN PATH - - - - ►
FATAL ———— ●
NON FATAL ———— o
REAR END COLLISION —►►
PARKED VEHICLE — ▫
FIXED OBJECT — □
OVERTURNED
OUT OF CONTROL
SIDES WIPE

TIME — A=AM, P=PM
PAVEMENT — D=DRY, I=ICY, W=WET
WEATHER — C=CLEAR, F=FOG, R=RAIN, SL=SLEET, S=SNOW

ACCIDENT SUMMARY	DAYLIGHT				NIGHT				TOTAL			
CLASSIFICATION BY TYPES	FATAL	NON FATAL	PROP. DAMAGE	TOTAL	FATAL	NON FATAL	PROP. DAMAGE	TOTAL	FATAL	NON FATAL	PROP. DAMAGE	TOTAL
APPROACHING AT RIGHTANGLE		1	2	3		1	1	2		2	3	5
APPROACHING SAME DIRECTION			1	1			1	1			2	2
APPROACHING OPP. DIRECTION							1	1			1	1
PEDESTRIAN ACCIDENTS		1		1						1		1
FIXED OBJECT ACCIDENTS												
OTHER ACCIDENTS												
TOTALS		2	3	5		1	3	4		3	6	9

Fig. 3.3. Collision Diagram

4. PARAMETERS OF INTERSECTION DESIGN

4.1. General

Intersections are designed having regard to flow speed, composition, distributio and future growth of traffic. Design has to be specified for each site with due regard to physical conditions of the site, the amount and cost of land, cost of construction and the effect of proposal on the neighbourhood. Allowances have to be made for space needed for traffic signs, lighting columns, drainage, public utilities etc. The preparation of alternative designs and comparison of their cost and benefits is desirable for all major intersections.

4.2. Design Speed

Three types of design speeds are relevant for intersection element design :

(i) Open highway or "approach" speeds

(ii) Design speed for various intersection elements. This is generally 40 per cent of approach speed in built up areas and 60 per cent in open areas.

(iii) Transition speeds for design of speed change elements i.e. changing from entry/exit speed at the intersection to merging/diverging speed.

The "Approach" speeds relevant to various types of terrain and roads are given in Table 4.1 and 4.2 respectively.

In rural areas ruling design speed should be used, but minimum can be adopted in sections where site conditions and costs dictate lower speeds. In urban areas a lower or higher value of design speed can be adopted depending on the pressure of physical controls, roadside developments and other related factors. A lower value is appropriate for central business areas and higher in sub-urban areas.

4.3. Design Traffic Volumes

Intersections are normally designed for peak hour flows. Estimation of future traffic and its distribution at peak hours is done on the basis of past trends and by accounting for factors like new development of land, socio-economic changes etc. Where it is not possible to predict traffic for longer period, intersection should be designed for stage development for design periods in steps of 10 yrs. Where peak hour flows are not available they may be assumed to be 8 to 10 per cent of the daily flow allocated in the ratio of 60: 40 directionally.

4.4. Radius of Curves at Intersection

4.4.1. The radii of intersections curves depend on the turning characteristics of design vehicles their numbers and the speed at which vehicles enter or exit the intersection area. The design curve is developed by plotting the path of the design vehicles on the sharpest turn and fitting curves or combination of curves to the path of inner rear wheels. Generally four types of curves are possible to fit in with the wheel paths of a turning vehicle.

Table 4.1. Design Speeds in Rural Sections (IRC : 73 - 1980)

Road classification	Design Speeds km/h							
	Plain	Terrain	Rolling	Terrain	Mountainous	Terrain	Steep	Terrain
	Ruling design speed	Minimum design speed	Ruling design speed	Minimum design speed	Ruling design speed	Minimum design speed	Ruling design speed	Minimum design speed
National and State Highways	100	80	80	65	50	40	40	30
Major District Roads	80	65	65	50	40	30	30	20
Other District Roads	65	50	50	40	30	25	25	20
Village Roads	50	40	40	35	25	20	25	20

Table 4.2. Design Speeds in Urban Areas

S.No.	Road Classifications	Design Speed (km/h)
1.	Arterial	80
2.	Sub-arterial	60
3.	Collection street	50
4.	Local street	30

	Type of Curve	Advantages/Disadvantages
(a)	Simple circular curve	Simple in layout but does not follow actual wheel path
(b)	3 - centered compound curve with offsets	Closest to actual wheel path and all transition curve but paved area is about 20 per cent more. Where the numbers of semi-trailer combination vehicles are substantial a symmetrical curve is closer to the wheel path.
(c)	Simple curve with offset and taper	This is closest to 3- centered compound curve and keeps paved area at intersection to a minimum
(d)	Transitional curves	Difficult to compute and layout but close to actual path.

The selection of any one curve depends on the types and sizes of vehicles that will be turning and decision on the extent to which a particular type of vehicle is to be used. The first step in design consists of study of the projected traffic data, the number and frequency of the larger units involved in turning movement and the effect of those large units on other traffic. If very large units are only occasional and they can turn with some encroachment on other traffic lanes and without disturbing traffic too much, it would be wasteful to design for the largest vehicle. However, the minimum design may need some modification to permit turning of the largest occasional vehicle.

4.4.2. Selection of appropriate curve radii, influences the vehicle speed at various points. The speed should be such that the vehicle should either be able to stop before the conflict point or accelerate to suitable speed to merge with traffic flow. The speed with which drivers can follow a curve can be taken to be $6\sqrt{R}$ km/h for upto 55 km/h, where R is the radius of curve in meters. Table 4.3 gives the relationship of inner curve radii for a larger range of design speeds.

Table 4.3. Design Speed & Minimum Radii

Design Speed km/hr	Minimum inner radii (m)
18.5	18
15	23
20	27
30	32
40	37
50	41
75	50
100	57
125	62
150	64
Straight	—

4.5. Design Vehicle

4.5.1. IRC : 3 - 1983 recognises three types of road design vehicles namely single unit truck, semi trailer and truck trailer combination. Passenger cars are not considered as design vehicles in rural areas as savings in construction using this vehicle cannot be justified on economic basis. As such nearly all intersection curves in rural areas should be designed for either single unit trucks/buses of 11/12 m length, or semi-trailer combination of 16 m length or truck-trailer combination of 18m length. On most rural highways semi-trailer combination would be used for design, whereas in non arterial urban areas a single unit truck or bus can form the basis for design. In purely residential areas, alone a car can form the basis of design.

4.5.2. The dimension and turning radii of some of the typical Indian vehicles are tabulated in Table 4.4. Dimensions and turning radii for typical vehicles viz. single unit truck, single unit bus, semi-trailer, large semi trailer and truck trailer combination trailer combination as per U.S. practice, are given in Table 4.5. In the absence of detailed investigations on Indian vehicles for their swept path etc., these vehicles are adopted for design purposes.

The swept path of different design vehicles and the selection of appropriate radii of turning circle at the intersection has been discussed in *Appendix I*.

4.5.3. There are five common situations in design of intersections and each one has to be generally designed for following conditions :

S.No.	Location of Intersection	Curve Design
1.	Rural Section	Design for single unit truck is preferred for intersection with local minor roads. Semi-trailer design is preferred for major road intersection where large paved areas result, channelisation also becomes essential.
2.	Suburban Arterial Section	Designed for semi-trailer with speed change lanes and channelisation. Three-centered compound curves are preferred.
3.	Urban Arterial & Sub-Arterials	Designed for single unit truck
4.	Urban Central Business Districts	Designed for single unit trucks for minimum curve radii with allowance for turning vehicles encroaching on other lanes.
5.	Residential area	Designed for cars only with encroachment of tracks into other lanes.

Table 4.4. Dimensions and Turning Radii of Some of the Typical Indian Vehicles

S.No.	Make of Vehicle	Length (m)	Width (m)	Turning Radius (m)
1.	Ambassdor	4.343	1.651	
2.	Maruti Car	3.300	1.405	4.400
3.	TATA (LPT 2416) 3-axled truck	9.010	2.440	
4.	TATA (LPO 1210) Full forward control Bus chasis	9.885	2.434	10.030
5.	TATA (LPO 1616) Bus chasis	11.170	2.450	
6.	Leyland Hippo Haulage	9.128	2.434	10.925
7.	Leyland (18746) Taurus	8.614	2.394	11.202
8.	Leyland Beaver Multi Drive	12.000	2.500	
9.	Mahindra Nissan Allwyn Cabstar	5:895	1.870	6.608
10.	Swaraj Mazda Truck (WT 49)	5.974	2.170	6.400
11.	DCM Toyota (Bus)	6.440	1.995	6.900

Table 4.5. Dimensions & Turning Radii of Design Vehicles

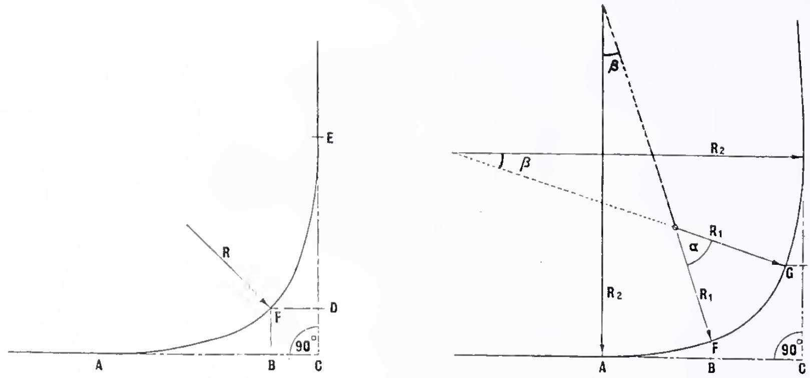
S.No.	Vehicle Type	Overall Width (m)	Overall Length (m)	Overhang Front (m)	Minimum Turning Rear (m)	Radius (m)
1.	Passenger Car (P)	1.4 - 2.1	3 - 5.74	0.9	1.5	7.3
2.	Single Unit Truck (S.U.)	2.58	9	1.2	1.8	12.8
3.	Semi Trailer and Single unit Bus (WB - 12 m)	2.58	15.0	1.2	1.8	12.2
4.	Large Semi-Trailer (WB-15 m)	2.58	16.7	0.9	0.6	13.71
5.	Large Semi-Truck Trailer (WB - 18 m)	2.58	19.7	0.6	0.9	18.2

4.6. Radii of Curves in Urban Situations

In urban areas additional conditions like restriction on right of way widths, abutting developments, pedestrian crossings, parked vehicles and high cost of land govern the minimum radii at intersections. Lower operating speeds and frequent signal stops also reduce the requirement of intersection areas. Generally, the minimum turning radius for a vehicle governs the design. However, to ensure efficient traffic operation on arterial streets a common radii of 4.5 m to 7.3 m for passenger cars and 9 m to 15 m for trucks and buses is recommended.

With use of single radius curve either a large radius must be used or width of street must be increased to accommodate larger vehicles. For this reason 3-centred curve or simple curves with offset or spirals to fit vehicle path should be used. The calculations for 3-centered compound curve symmetrical and assymetrical are given in Table (I-3 to I-5) of *Appendix - I* to fit the edge of pavement closely to the minimum path of the design semi-trailier combinations, assymetrical arrangement of 3 - centred compound curves should be used. However if smaller vehicles make up a sizeable percentage of turning volume, simpler symmetrical arrangement should be adopted. A suitable curve design in urban areas is shown in Fig. 4.1. Where special parking lanes are provided, most vehicles are able to turn without encroaching on adjacent lanes, i.e. by using the parking lane, as shown in Fig. 4.2. Parking must, however, be restricted for a distance of at least 4.5 m in advance of start of curve on the approach and 9 m beyond the end of curve at the exist. For bus and WB-15.0 vehicles, the parking prohibition of exit should be 12 m.

In urban areas, if curve radii is increased, the pedestrian crossing distance increases as shown in Fig. 4.3. Since this has pedestrians safety implications, this should be kept in view while deciding on the turning radius to be provided.



(A) TWO SPIRALS

(R = 6m, 9m, 10.5m)

	AC/CE m	AB/DE m	BF/DF m
10.5 m	19.140	15.190	3.950
9 m	16.830	13.290	3.540
6 m	11.220	8.860	2.360

(B) COMPOUND CURVE

(R = $2R_1$ TO $3R_1$)

R_1	R_2	β	α	AC/CE	AB/DE	BF/DC
10.5	31.5	18°	54°	17.5 m	9.33°	1.42
9	27	18°	54°	15 m	8°	1.22
6	18	18°	54°	10 m	5.33°	0.81

Fig. 4.1. Design of Street Lanes Curve

4.7. Width of Turning Lanes at Intersection

Determination of widths of turning lanes at intersection is primarily based upon the type of vehicles using it, the length of lanes, the volume of traffic and if kerbs are provided, the necessity to pass a stalled vehicles. Table 4.6 gives the recommended widths of turning lanes. These can be assumed to have a capacity of 1200 PCU/hr.

Table 4.6. Width of Lanes at Intersections

Inner Radius	Design speed km/h.	Single lane width m	Single lane width with space to pass stationary vehicles m	Two lane width for one or two way traffic m
(1)	(2)	(3)	(4)*	(5)
10.5	18	5.50	10.53	11.5
15	23	5.50	9.50	10.5
20	27	5.00	9.00	10.0
30	32	4.50	8.00	9.0
40	37	4.50	7.50	9.00
50	41	4.50	7.00	8.00
75	50	4.50	7.00	8.00
100	57	4.50	7.00	8.00
125	62	4.50	6.50	8.00
150	64	4.50	6.50	8.00
—	—	4.50	6.00	7.00

* These widths are applicable for longer slip roads (over 60m length) and should be used only if vehicles are allowed to park.

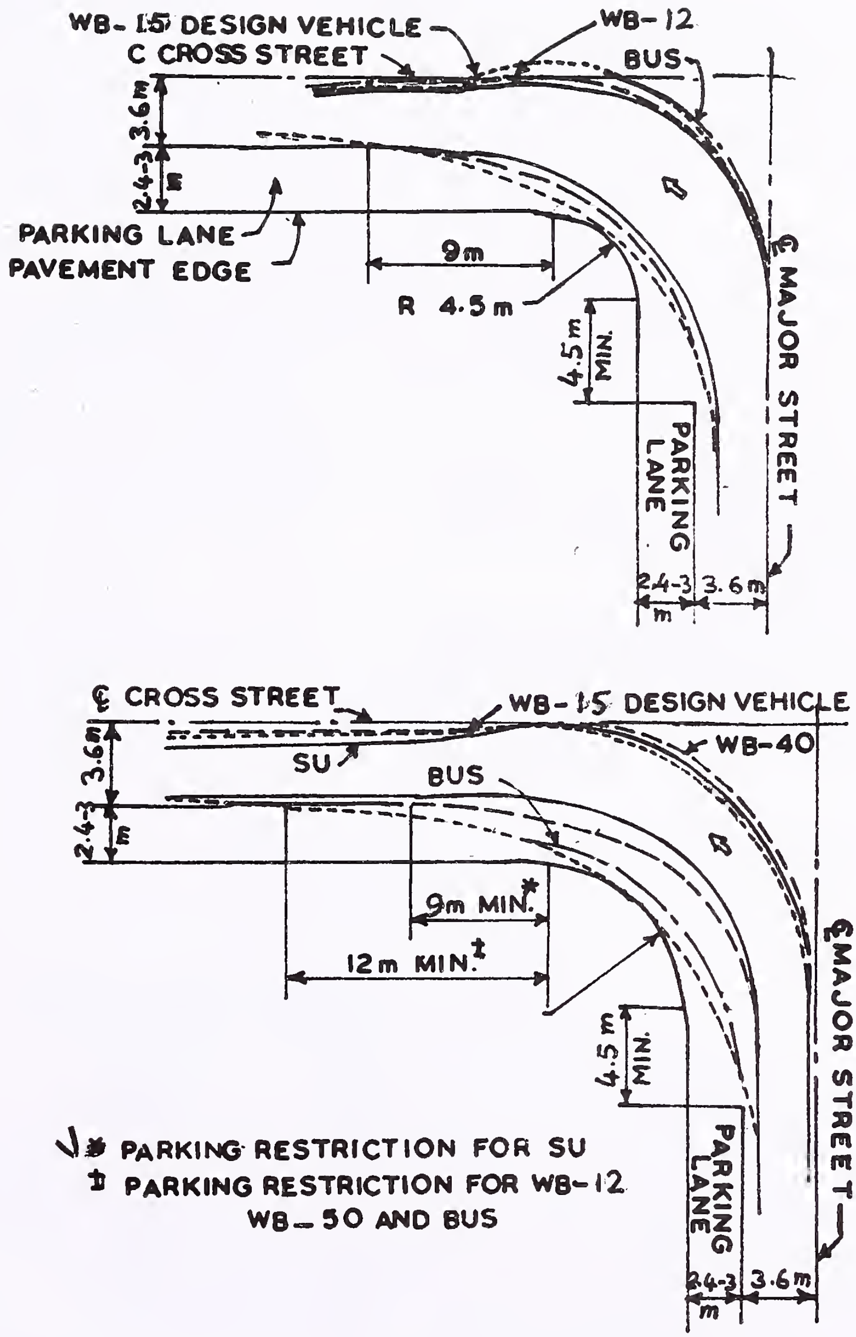
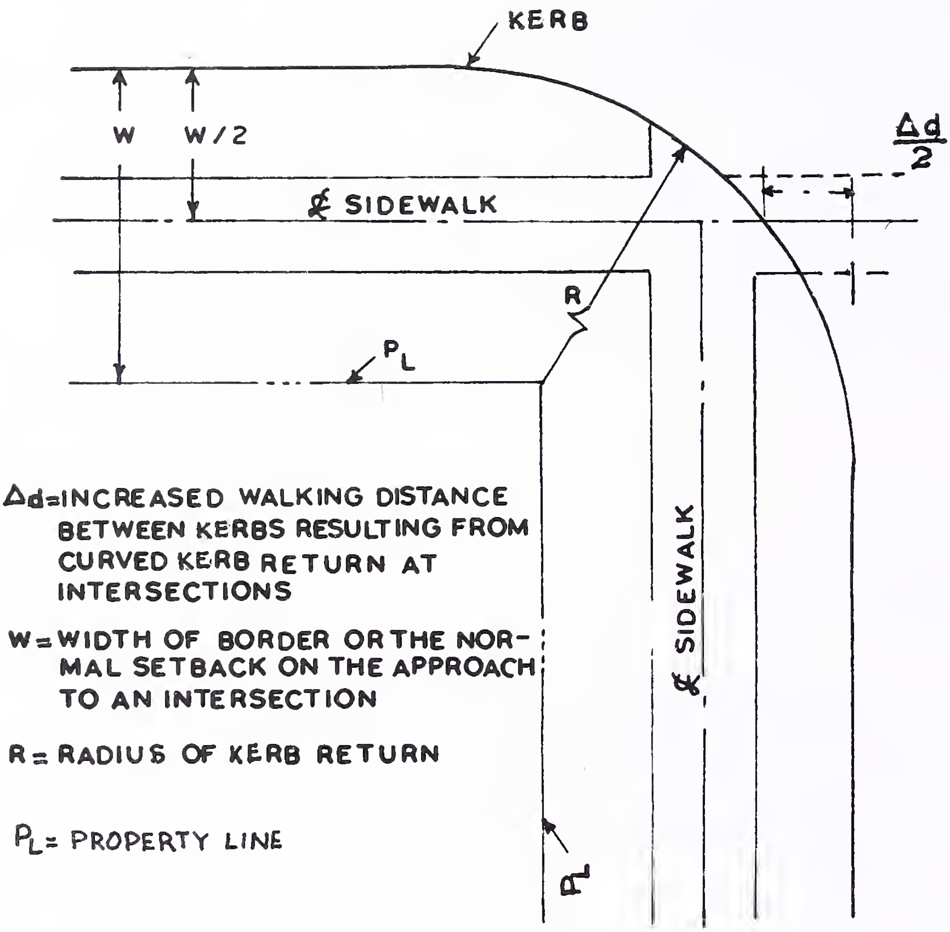


Fig. 4.2. Effect of Kerb Radii and Parking on Turning Paths



ADDED CROSSWALK DISTANCE Δd

KERB RADIUS, R (m)	W=3 m (m)	W=6 m (m)
3	1	0
6	4.2	1.5
9	8.1	4.5
12	12.6	8.1
15	17.1	12

Fig. 4.3. Variations in Length of Crosswalk with Corner-Kerb Radius and Width of Border

4.8. Auxiliary Lanes

Three types of auxiliary lanes are provided at intersections. These are storage lanes, right turning lanes, acceleration lanes and deceleration lanes. The last two together are also called speed change lanes. Provision of these increases the capacity of intersection and improves safety. The length of these lanes depends on the volume of traffic entering or leaving the side road. The shape of these can be either parallel lane with sharp taper or a direct taper or with a transition curve. Fig. 4.4 shows the method of introducing addition lane using transition curves.

4.8.1. **Storage lanes/right turning lanes :** Storage lanes are generally more important in urban areas where volume of right turning traffic is high and if not catered for, blocks the through traffic. Normal design procedure provides for storage length based on 1.5 times the average number of vehicles (by vehicle type) that would store in turning lane at peak hour. At the same time the concurrent through lane storage must also be kept in view, as it may happen that entry to turning lane may become inaccessibile due to queued vehicles in through lane. Fig. 4.5 shows several methods of introducing turning lane at intersections. Figs. 4.6 and 4.7 show satisfactory method of widening at intersections and widening for turning lanes at intersections.

Example of Design

Consider an urban intersection with angle, right turn lane and the through lanes with a signal installed with a cycle time of 90 sec. Assume the right turn traffic volume at design peak hour to be 150 vehicles (10 trucks) and through volume to be 520 vehicles (15% trucks). The storage lane length is determined as follows :

$$\text{No. of cycles per hour} = \frac{60 \times 60}{90} \times 40$$

$$\text{No. of right turning vehicles per cycle} = \frac{150}{40} = \text{say } 4$$

$$\text{and No. of through vehicles per cycle per hour} = \frac{520}{40 \times 2} = \text{say } 7$$

Assuming the peak traffic within the hour to be uniform (if this is not so further adjustment will be required) the length of lanes, using a truck adjustment factor and car length of 7.5 m (Ambassador car) and truck length of 11m, lane length is determined as below.

Length of right turn lane = 4 x 0.9 x 7.5 + 4 x 0.1 x 11 = 31.4 m

Length of through lane = 7 x 0.85 x 7.6 + 7 x 0.15 x 11 = 56.17 m

Choose design length of storage lane = 56 m.

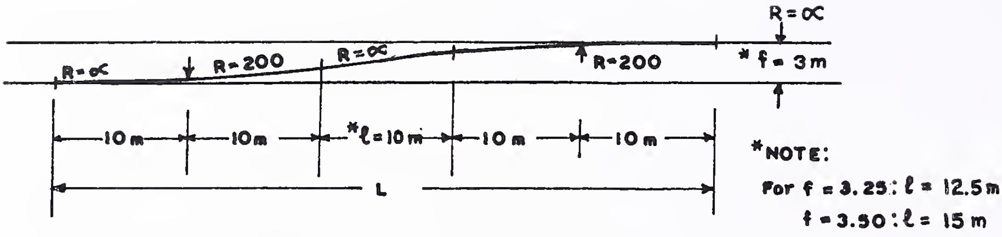


Fig. 4.4. Method of Transition Curves at Points of Additional Lane

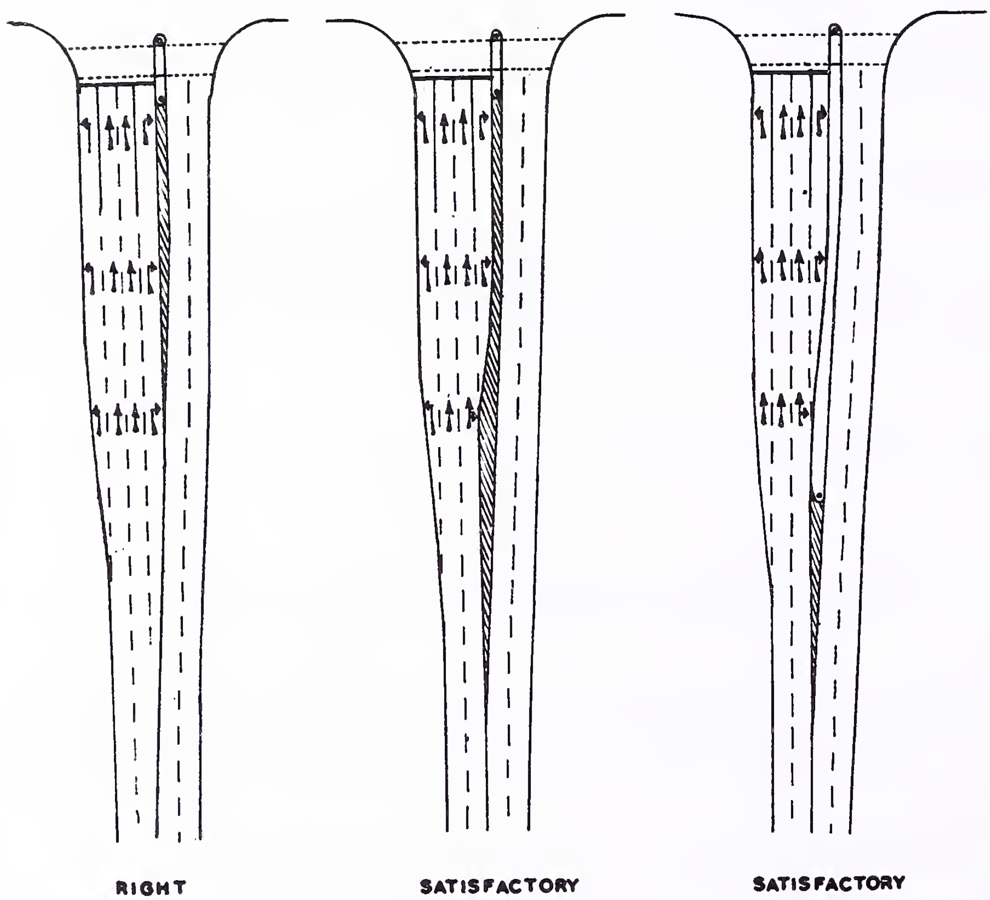
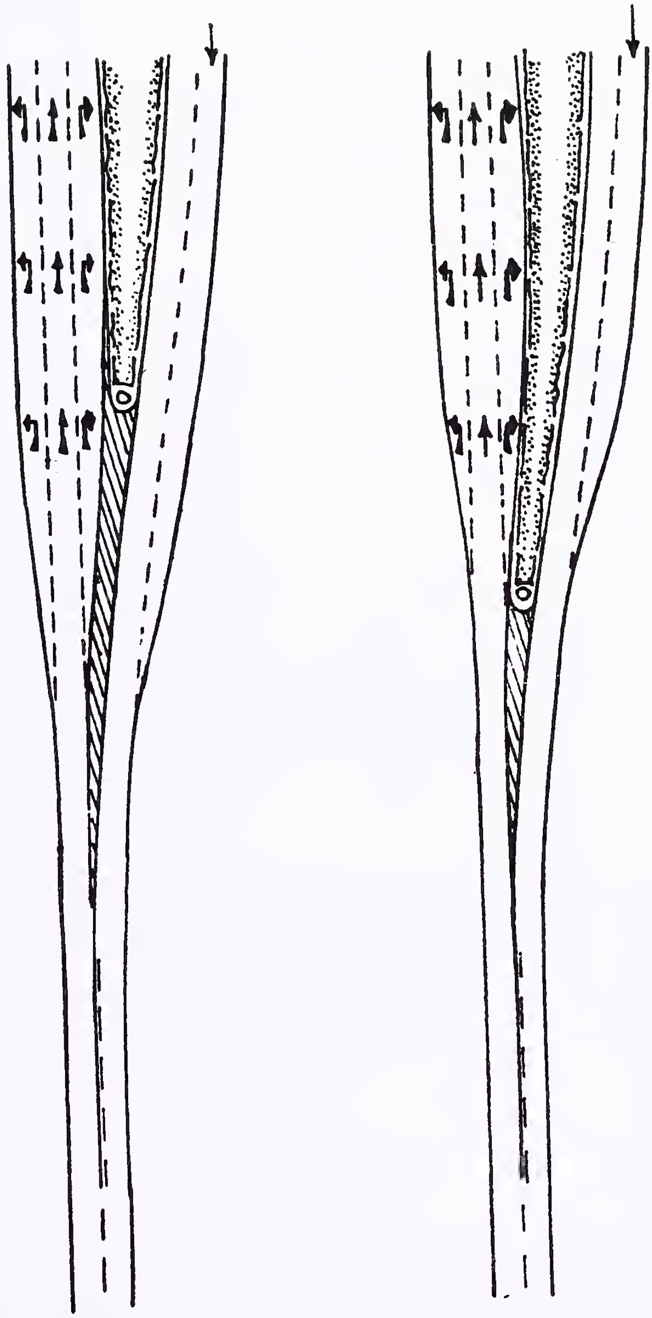


Fig. 4.5. Provision of Turning Lanes at Intersections



BETTER SATISFACTORY

Fig. 4.6. Method of Widening of Intersections

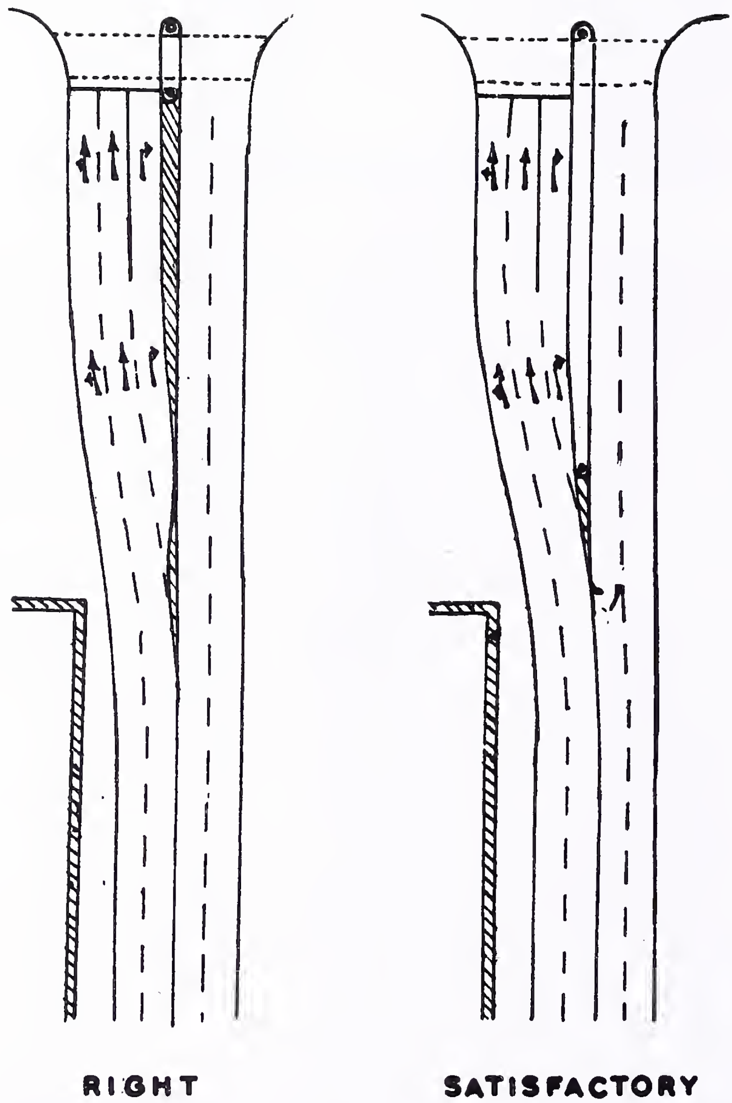


Fig. 4.7. Method of Widening for Turning Lanes at Intersections

In places where not more than one or two vehicles are expected to wait for right turn, such as in rural areas, the storage lane may be provided as per Table 4.7.

Table 4.7. Length of Right Turning Lane

Design Speed (km/h)	Length of storage lane including 30 - 45 m taper (m)
120	200
100	160
80	130
60	110
50	90

4.8.2. **Speed Change Lanes :** Speed change lanes are more important in rural areas. In urban areas such lanes are rarely required but provision of short lanes to assist merging and diverging manoeuvres are provided in conjunction with channelising islands. Speed change lanes should are uniformly tapered and have a set back of 5.4 m at the tangent point of curve leading into or out of minor road. The turning lane should be reduced in width to 4.25 m by carriageway marking etc. as shown in Fig. 4.8.

Acceleration lanes

An acceleration lane should be designed so that vehicles turning left from the minor road may join the traffic flow on the major road at approximately the same speed as that of the nearside lane traffic in the major road. Acceleration lanes also improve capacity by enabling the use of short traffic gaps and by providing storage space for traffic waiting to merge when large traffic gaps occur. Acceleration lanes are recommended where the future traffic on the acceleration lane is accepted to be more than 1,000 PCU's per day.

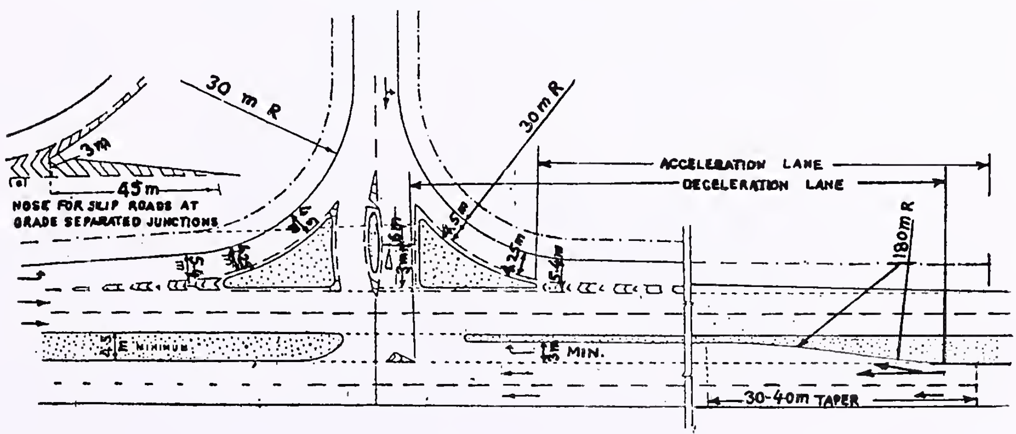


Fig. 4.8. Typical Dimensions of Road Intersections

Recommended lengths of acceleration lanes for different main road design speeds are given in Table 4.8 and a typical layout is given in Fig. 4.8. In difficult conditions sub-standard lengths may have to be accepted, but these should not be less than half of those recommended.

Table 4.8. Minimum Acceleration Lane Lengths

Highway		Acceleration Length (m) for entrance curve design speed (kmph)								
		Stop conditions	25	30	40	50	60	65	75	80
Design Speed (kmph)	Speed Reached (kmph)	and initial speed (kmph)								
		0	20	30	35	40	50	60	65	70
50	40	60	-	-	-	-	-	-	-	-
65	50	120	100	75	70	40	-	-	-	-
80	60	230	210	190	180	150	100	50	-	-
100	75	360	340	330	300	280	240	160	120	50
110	85	490	470	460	430	400	380	310	250	180

Table 4.9. Minimum Deceleration Lane Length

Highway Design Speed (kmph)	Average Running Speed (kmph)	Deceleration Length (m) For Design Speed of Exit Curve								
		Stop condition	25	30	40	50	60	65	75	80
		for Average Running Speed of Exit Curve								
		0	20	30	35	40	50	60	65	70
50	45	70	60	50	40	-	-	-	-	-
65	60	95	90	80	70	60	50	-	-	-
80	70	130	120	120	110	100	90	70	50	-
100	85	160	150	150	140	130	125	100	90	70
105	90	175	165	160	150	150	130	120	100	85
110	95	190	180	175	170	160	150	130	120	100

Where acceleration lanes are on a down gradient their length may be reduced to 1-0.08G times the normal length, where G is the gradient expressed as a percentage.

Deceleration Lanes

Deceleration lanes are of greater value than acceleration lanes because the driver of a vehicle leaving the highway has no choice but to slow down any following vehicles on the through lane if a deceleration lane is not provided. Deceleration lanes are needed on the near side for left turning traffic and on the right turn lane where provision is made for right turning traffic.

The length of near side deceleration lanes should be sufficient for vehicles to slow down from the average speed of traffic in the near side lane to the speed necessary for negotiating the curve at the end of it; in order to make deceleration lanes effective, the curve radius must permit a speed of at least 30-40 kmph (not less than 30 m). Recommended lengths of near side deceleration lanes are given in Table 4.9 and a suitable layout is given in Fig. 4.8. Near side deceleration lanes are recommended for intersection on roads where the future traffic on the deceleration lane is expected to be more than 750 p.c.u's/day.

Where the number of traffic lanes on a road is reduced immediately beyond a slip road, in order to avoid entrapping through vehicles in the slip road the carriageway should be constructed to full width to the exit nose and a taper length of 180 m provided beyond it.

Right-turn deceleration lanes in the central reserve should be provided at all gaps for right-turning traffic on dual-carriageway roads. On three-lane roads, the centre lane should be marked for right turning traffic where the product of estimated future cutting flows in p.c.u's/per day is more than one million. The widening of two-lane single-carriageway roads to provide right-turn deceleration lanes in the centre of the road should be considered at the same levels of flow as for three-lane roads. These provisions may be made for lessser flows where accident records warrant them, or on two-lane roads where they can readily be incorporated in realignment or other scheme. On overloaded three lane roads or where the road junction is on a crest, it is usually desirable to construct short lengths of dual carriageways and provide right-turn deceleration lanes for right-turning traffic.

The lengths of right-turn deceleration lanes should be sufficient for vehicles to slow down to a stop from the average speed of vehicles in the off side lane omission of these lanes will usually result in numerous head to tail collisions. These lanes should not be less than 3 m wide and parallel-sided with entry and return radii of 180 m giving a taper of 30 - 45 m.

Even if it is not practicable to provide the full length of deceleration lane (right-turn or nearside) sub-standard lengths are still of great benefit but they should not be less than half the recommended lenghts.

Where deceleration lanes are on an up-gradient their length may be reduced to that obtained by multiplying the recommended length by 1 — 0.03G whereas G is the gradient expressed as a percentage. For deceleration lanes on a down gradient their length may be increased that obtained by multiplying the recommended length by 1 + 0.06G.

4.9. Super Elevation and Cross-slope

Where the turning slip lanes are provided for higher speed operation at intersection, they should be superelevated for the appropriate speed as given in the appropriate geometric design standard (Fig. 4.9) The principle of superelevation runoff also applies. But in intersection design the actual curves are of limited radii and length. As such in practice it is difficult to provide the required superelevation without causing abrupt cross-slope change, which could be dangerous. In practice therefore lower rates of superelevation are often accepted to intersections to maintain riding comfort, appearance and to effect a balance in design. The cross slopes in the intersecting area should be maintained as per IRC : 73 - 1980. In the intersection area normally the pavement cross-slope should be carried through to the turning lanes as well to avoid creation of drainage problem.

Extreme care must be exercised to check the drainage of the entire intersection area and cross-over crown lines. Where necessary drainage inlets should be designed and so located as to minimise the spread of water on traffic lanes and eliminate stagnant pools in the intersection area. No sheet flows should be allowed across the intersection where pavement surface are warped and surface water should be intercepted before the change in cross-slope. Also inlets should be located upgrade of pedestrian crossing so that the pedstrian crossings are always free of water.

At turning lanes of intersection, superelevation commensurate with radii and speed is seldom possible as too great a difference in cross-slope may cause vehicles changing lanes and crossing crown line to go sideways with possible hazard. When high-bodied trucks cross the crown line at some speed at an angle of about 10° to 40°, the body throw may make vehicle control difficult and may result in overturning. The method of developing superelevation of turning lanes for different situations is illustrated in Fig. 4.10 to 4.13.

From safety consideration the maximum algebric difference in pavement cross slope of various intersecting areas should be limited to figures given in Table 4.10 and transition cross-slope should be used.

Table 4.10. Maximum Algebric difference in Pavement Cross Slope

Design Speed km/hr	Maximum algebric difference of cross slope over cross line	
	75 per cent Trucks	5 per cent Trucks
24	0.060	0.080
32	0.055	0.075
40	0.050	0.065
40	0.045	0.055
56	0.040	0.050
Over 56	0.040	0.045

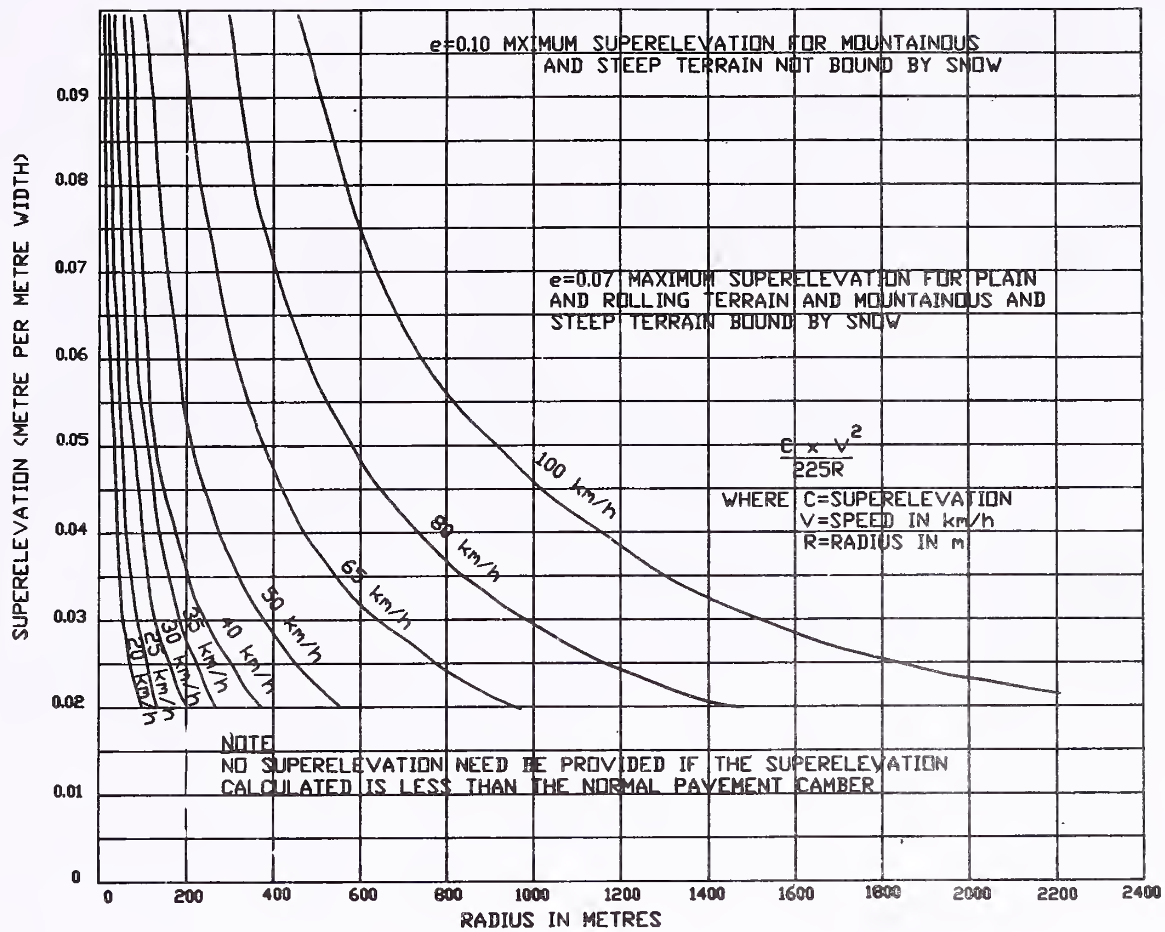
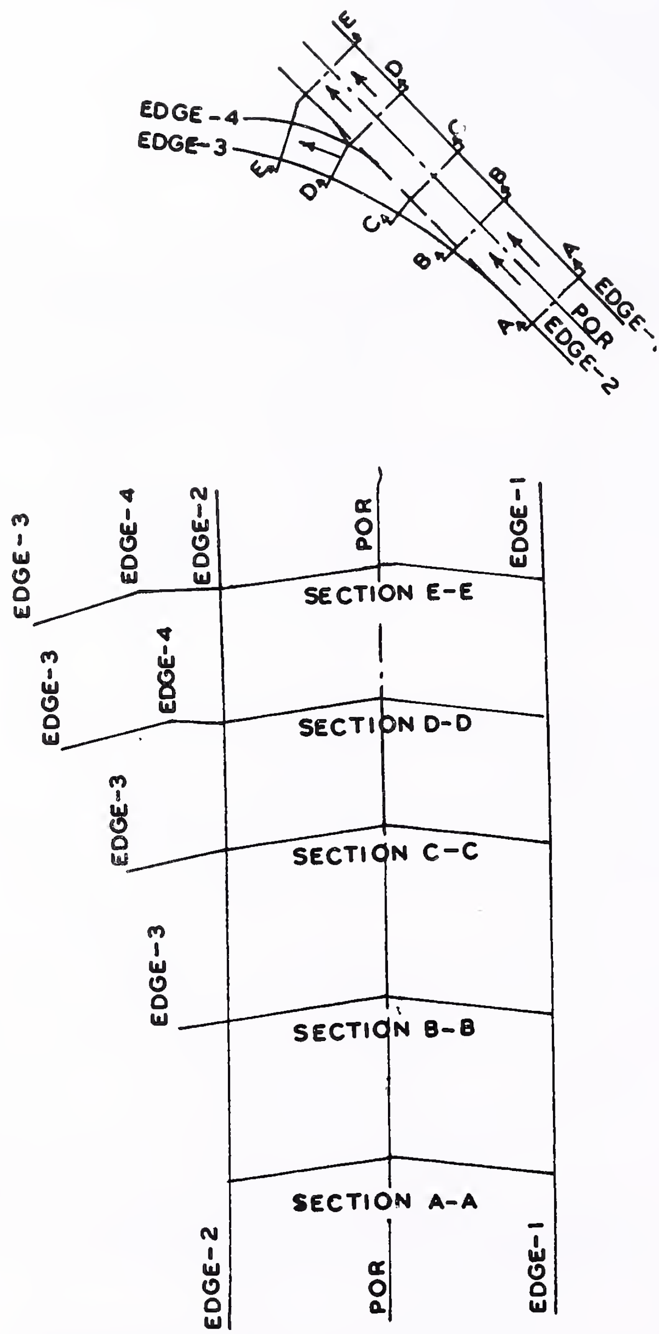
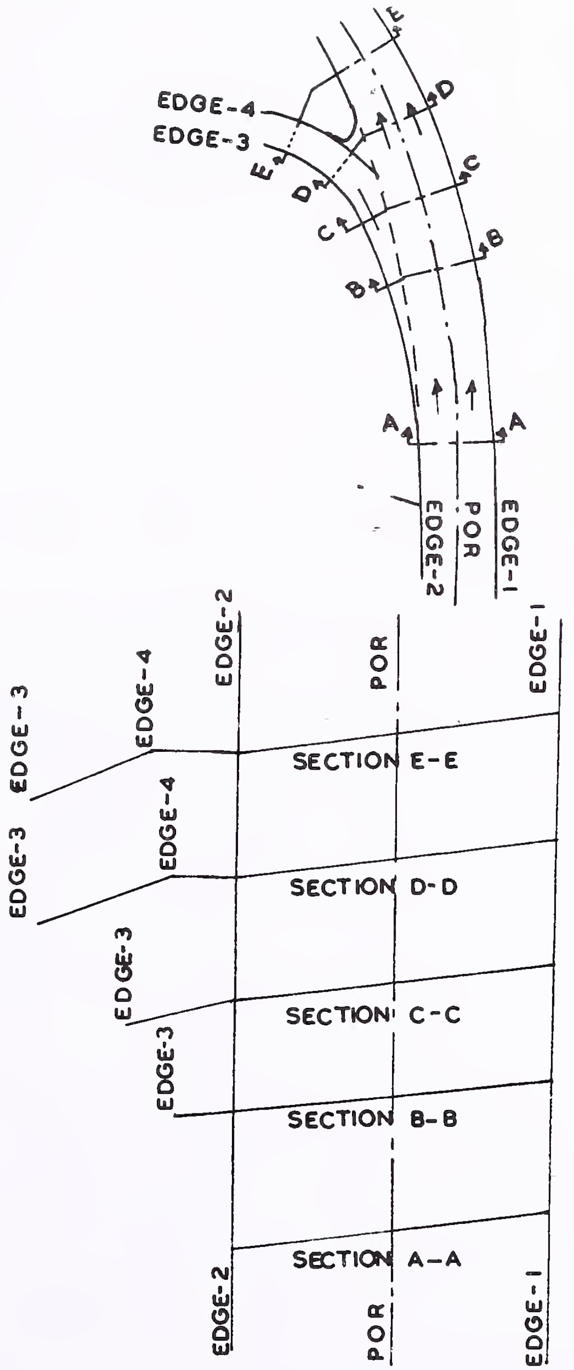


Fig. 4.9. Superelevation Rates for Various Design Speeds



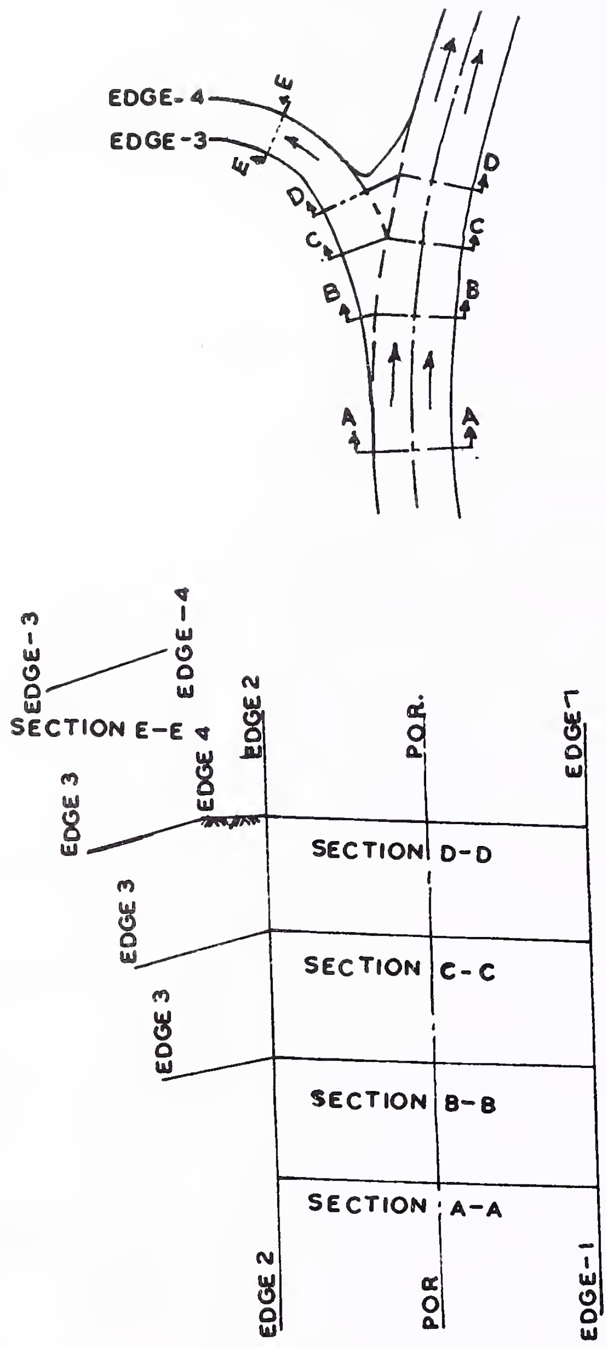
NOTE: POR = POINT OF ROTATION

Fig. 4.10. Development of Superelevation at Turning Roadway Terminals



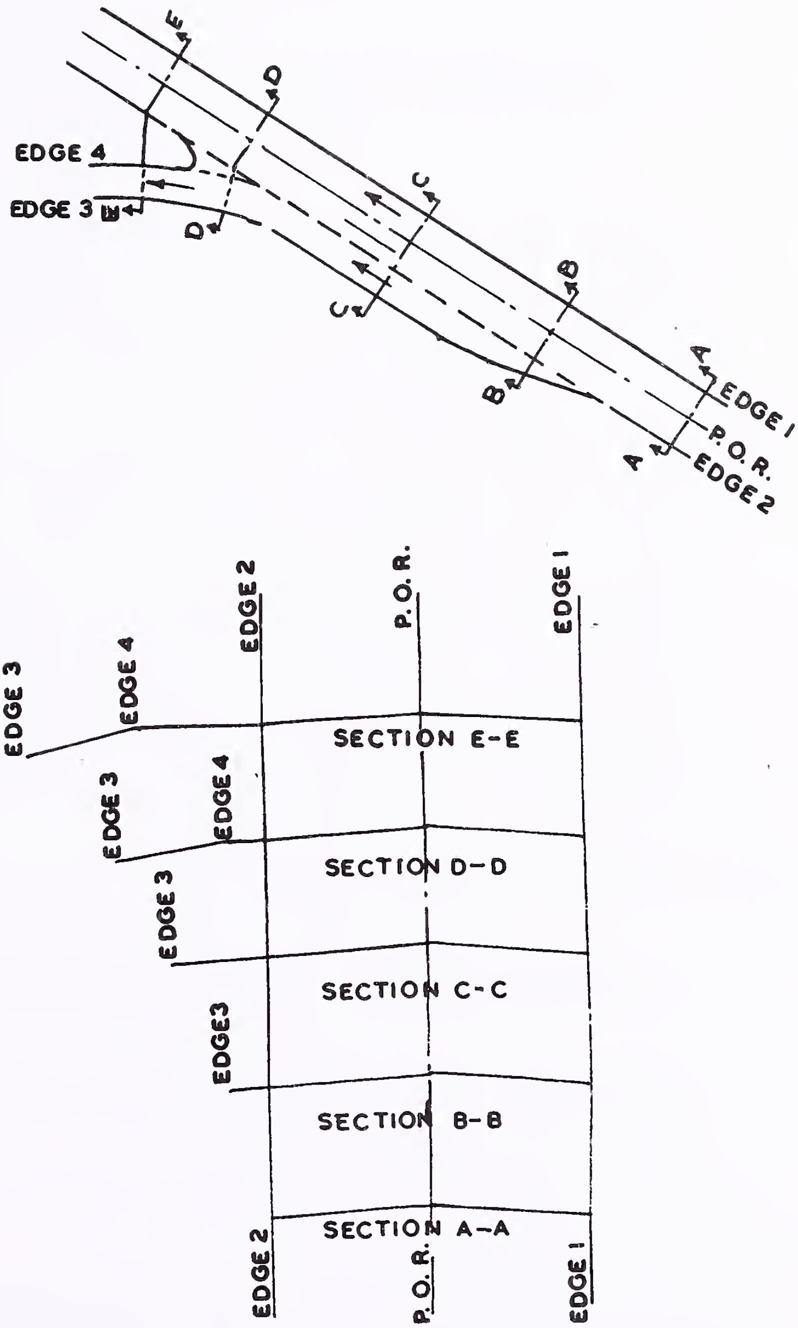
NOTE: POR = POINT OF ROTATION

Fig. 4.11. Development of Superelevation at Turning Roadway Terminals



NOTE: POR = POINT OF ROTATION

Fig. 4.12. Development of Superelevation at Turning Roadway Terminals



NOTE: P O R = POINT OF ROTATION

Fig. 4.13. Development of Superelevation at Turning Roadway Terminals

4.10. Visibility at Intersections

4.10.1. The sight distance is one of the major factor in safety at intersections. There are two considerations which are important to the driver as he approaches an intersection :

(i) Overall visibility at intersection layout so that it can be comprehended properly at first glance by the approaching driver, for visualising the prospective worthiness of the layout, a simple method for this is to hold the junction drawing horizontally at eye level and observe the proposed layout from the direction of each approach, simulating the drivers view of the junctions. This squinting procedure can remarkably bring out many defects in the design.

(ii) Sight triangle visibility to negotiate an intersection is another important requirement on becoming aware of approaching intersection, the driver must be able to observe and comprehend the speed and direction of approaching traffic from all other legs of the intersection. If a vehicle is approaching he should be able to safely stop prior to reaching the intersection. The approaching driver must be able to see sufficient distance along the cross road so as to judge if he can cross by suitably adjusting the speed and direction. Special care to ensure visiblity should be taken if intersection is located on high land in a cutting at or near a summit or near a bridge. Telephone poles, kiosks, signs, lightposts etc. should not be placed where they restrict visibility.

4.10.2. IRC : 66 - 1976, identifies two specific intersection conditions that are relevant to minimum sight triangle. These conditions are :

(i) "Uncontrolled intersections" where the intersecting roads are of more or less equal importance and there is no estabilished priority.

(ii) "Priority intersections" like minor road intersections where one road takes virtual precedence over the other. Traffic on minor road may be controlled by Stop or Give way signs/road markings.

Sight distance requirements in both these cases have been illustrated in Figs. 4.14 & 4.15.

4.10.3. The stopping sight distance required at uncontrolled intersections for different vehicles speeds is given in Tables 4.11 on the next page :

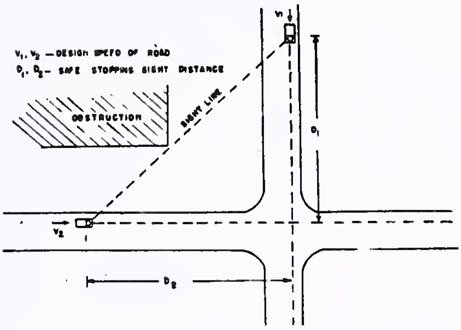
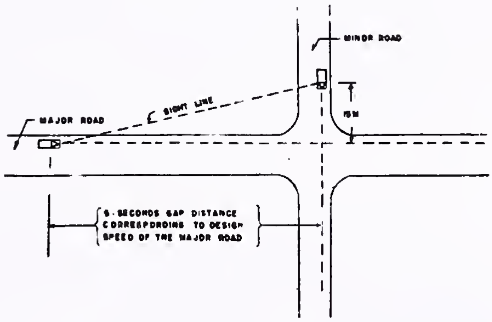


Fig. 4.14. Minimum Sight Triangle at Uncontrolled Intersections



Note :- Any obstruction should be clear of the minimum visibility triangle for a height of 1-2 m above the roadway.

Fig. 4.15. Minimum Sight Triangle at Priority Intersections

Table 4.11. Safe Stopping Sight Distance of Intersections

Speed	Safe stoping Sight Distance (m)
20	20
25	25
30	30
40	45
50	60
60	80
65	90
80	130
100	180

For priority intersections IRC : 66-1976 recommends a minimum visibility of 15 m along the minor road while for the major road, sight distance equal to 8 seconds travel at design speed is recommended. Visibility distances corresponding to 8 seconds travel time are set out below in Table 4.12.

Table 4.12. Visibility Distance on Major Roads

Design Speed (km/h)	Minimum Visibility Distance along major road (m)
100	270
80	180
65	145
50	110

All sight distance obstructions, like bushes, trees and hoardings in the visibility triangle should be removed to improve safety. Fig. 4.16 shows the desirable amount of trimming to be done to hedges and trees at intersections.

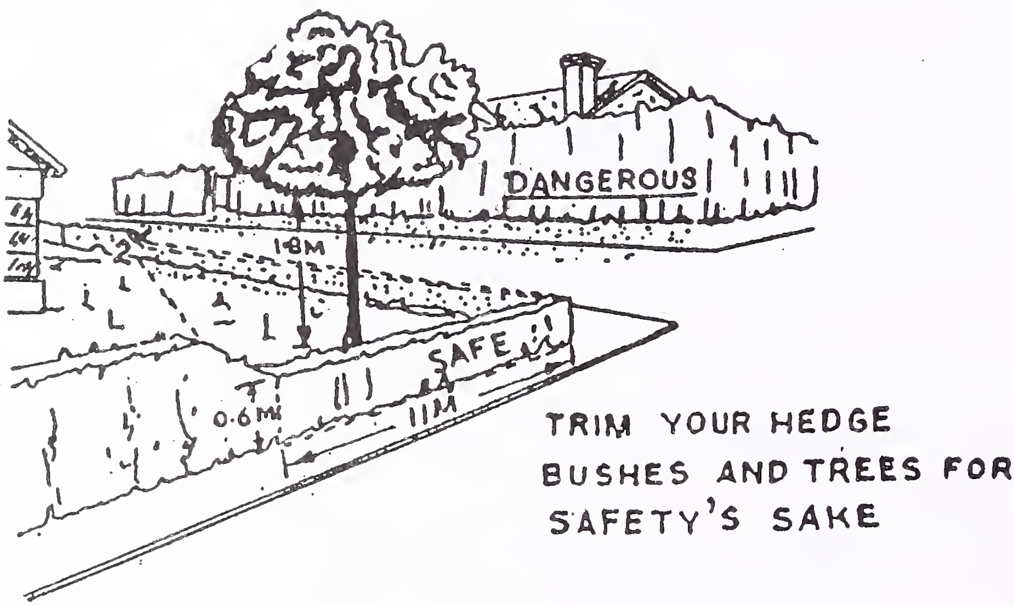


Fig. 4. 16. Trimming of Trees and Hedges Required for Clear Sight Distance

4.11. Channelising Island

4.11.1. The objectives of providing channelising island are to :

(i) control speed and path of vehicles at the intersection ;
(ii) control angle of conflict ;
(iii) separate conflicting traffic streams ;
(iv) provide shelter to vehicles waiting to carry out certain manoeuvres;
(v) assist pedestrians to cross;
(vi) reduce excessive carriageway areas and thus limit vehicle paths; and
(vii) locate traffic control devices.

The general types of island and their shapes are shown in Fig. 4.17. To ensure proper functioning of each type of islands, principles given below for each should be adhered to

4.11.2. **Corner or directional islands** : Figures 4.18 to 4.20 illustrate the design features of corner islands and the considerations which govern their sizes and shapes. Corner or Directional Islands (normally triangular) should meet the following requirements :

(a) It should be of sufficient size to be readily identified and visible. For an island to be clearly seen it must have an area of at least 4.5 m^2 in urban areas and 7 m^2 in rural areas and should usually be bordered with painted raised kerbs. Smaller areas may be defined by pavement marking Accordingly triangular islands should not be less than 3.5 m and preferably 4.5 m on a side after rounding of curves.

(b) It should be offset from normal vehicle path by 0.3 m to 0.6 m. The layout should be tested using the track diagram for all turning movements (Fig. 4.21).

(c) It should be provided with illuminated sign or a ballard at suitable places e.g. apexes of islands. It should be of sufficient size to enable placement of such traffic control devices.

(d) It should be accompanied by suitable carriageway marking to show actual vehicle paths. Marking should be made conspicuous by use of reflectorised materials.

(e) It should be properly marked for night visibility.

4.11.3. **Centre or divisional islands** : Centre islands requires careful location and designing. They require careful alignment and are invariably accompanied by widening of right-of-way as illustrated in Fig. 4.22. Centre or divisional islands should meet the following requirements

(a) It should be preceeded by a clearly marked or constructed natural area of not less than 1.5 sec. travel time at approach speed (Fig. 4.23).

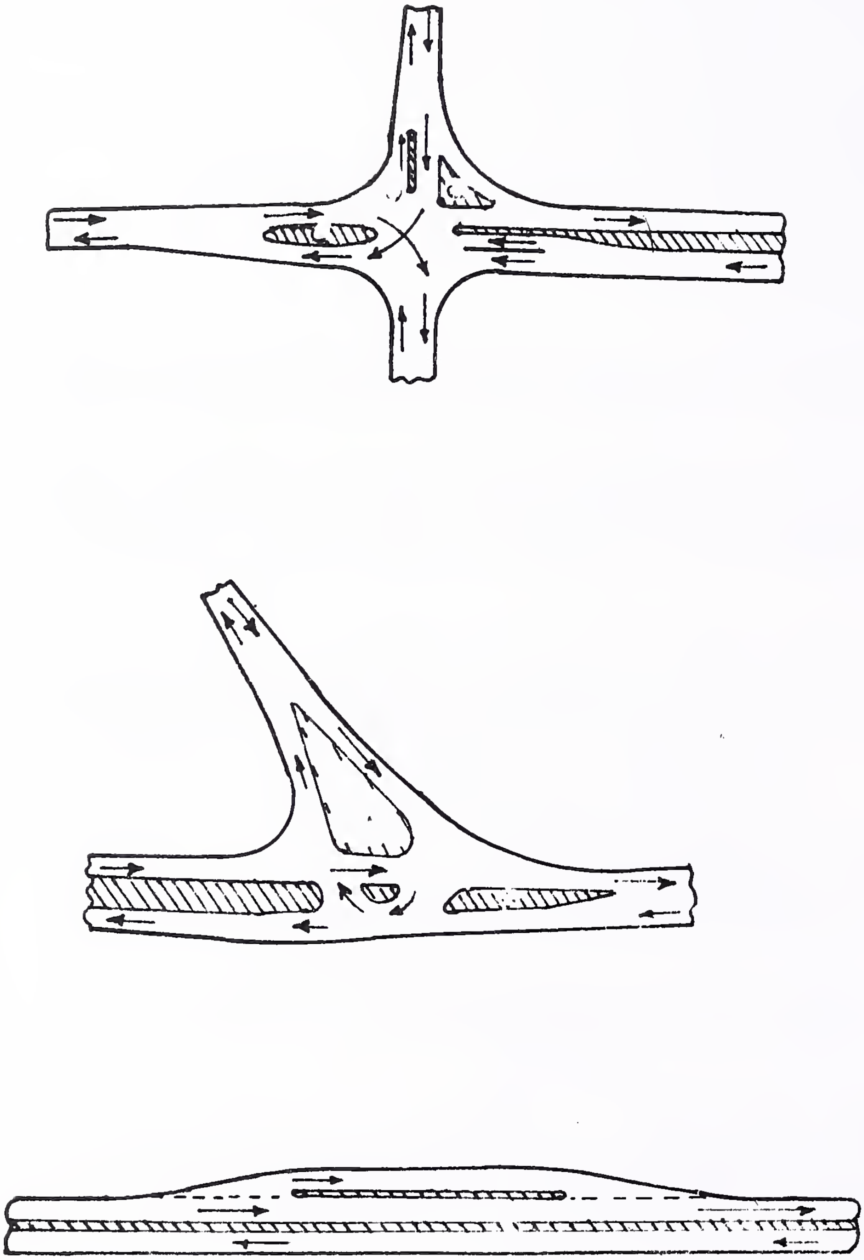
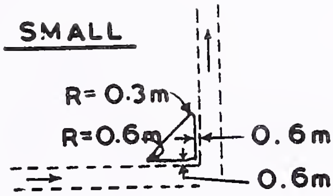
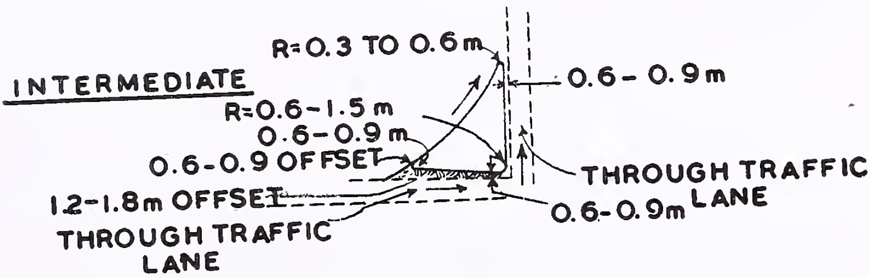


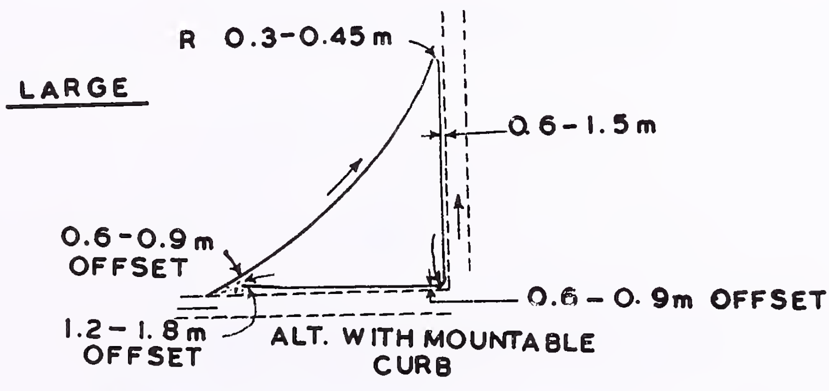
Fig. 4.17. General Types and Shapes of Islands



NOTE:
LAYOUTS SHOWN ALSO APPLY TO LARGE AND INTERMEDIATE ISLAND WITHOUT KERBS, ISLAND SIDE OFFSETS DESIRABLE BUT MAY BE OMITTED

PAINTED STRIPES CONTRASTING SURFACE, JIGGLE BARS, ETC.





KERBED ISLANDS - NO SHOULDER

Fig. 4.18. Details of Triangular Island Design (kerbed islands, no shoulder)

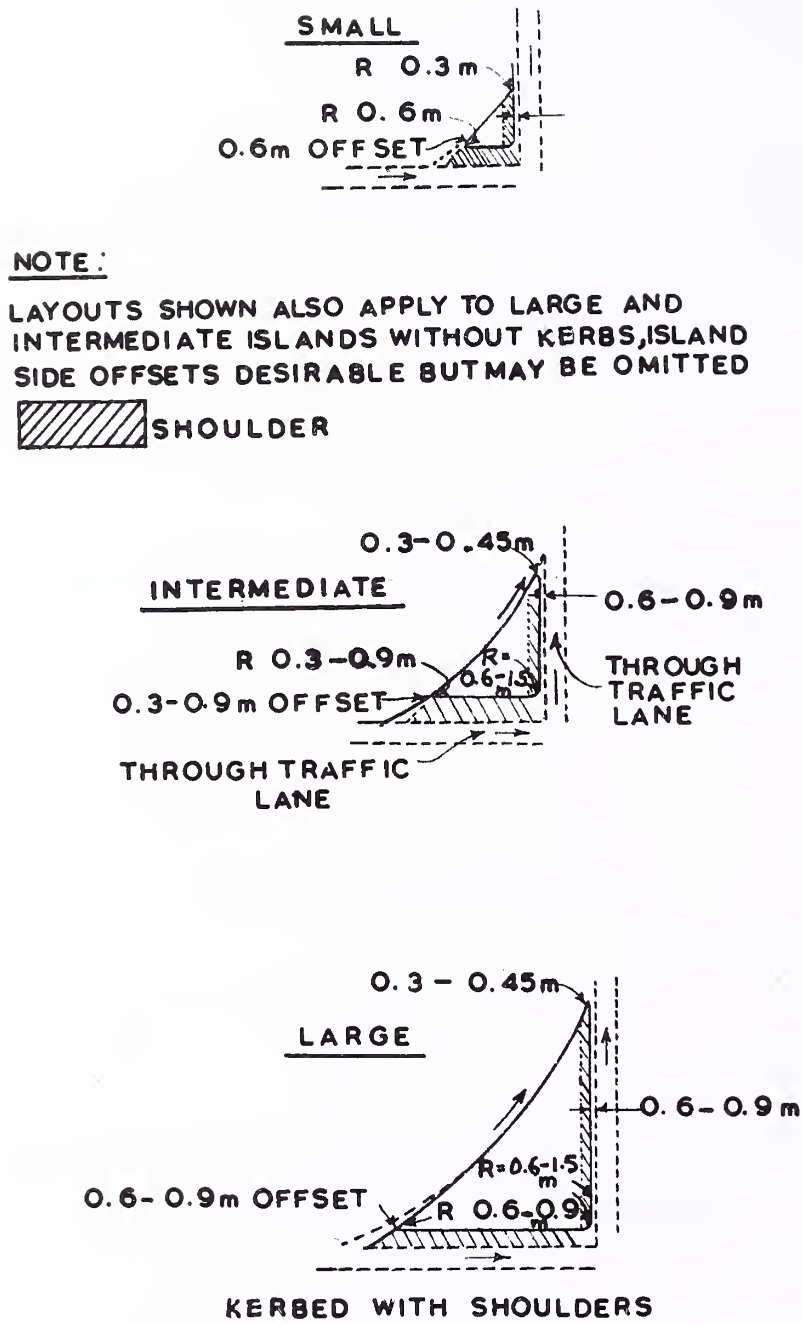


Fig. 4.19. Details of Triangular Island Design (kerbed island with shoulders)

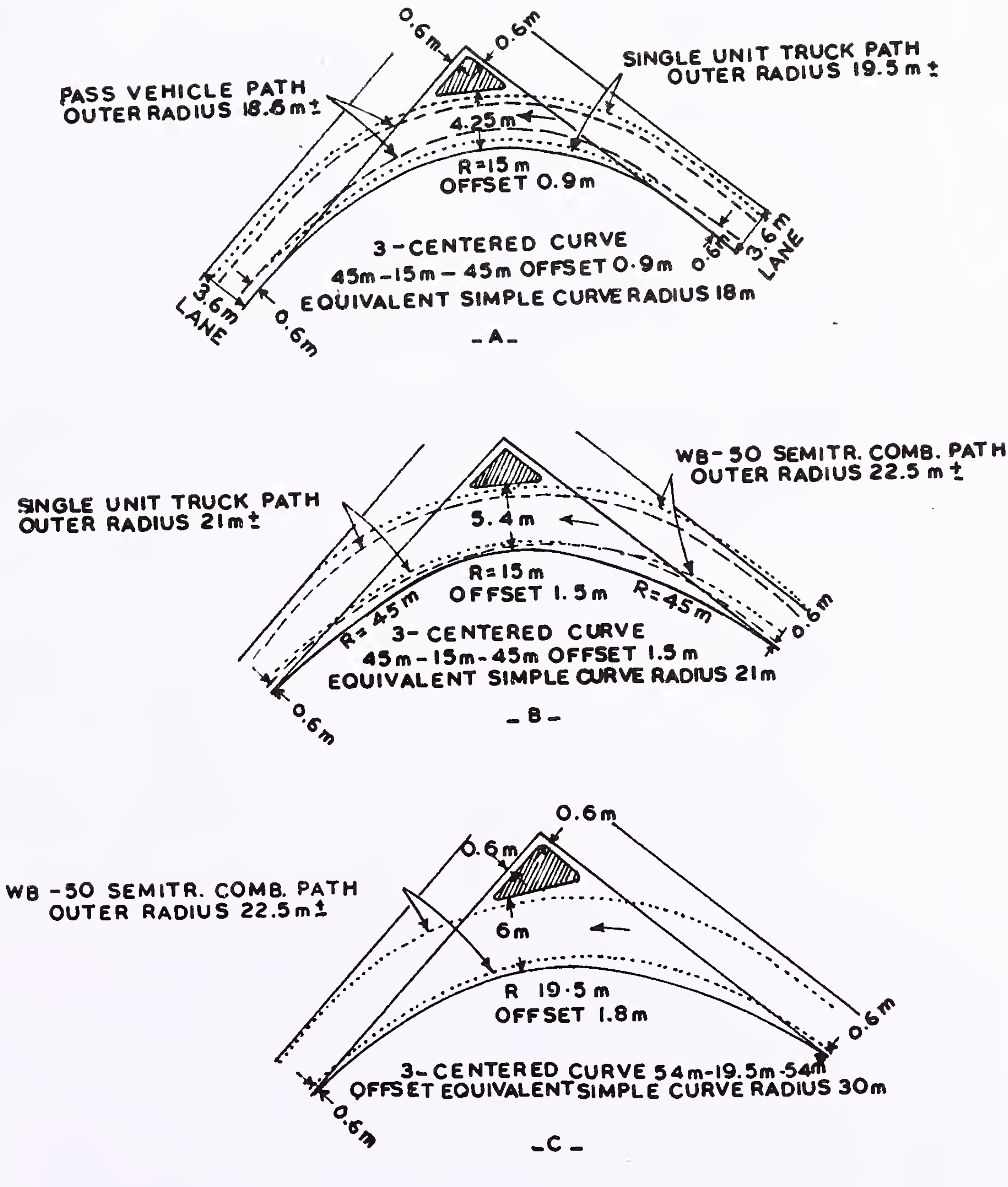


Fig. 4.20. Design for Turning Roadways with Minimum Corner Island

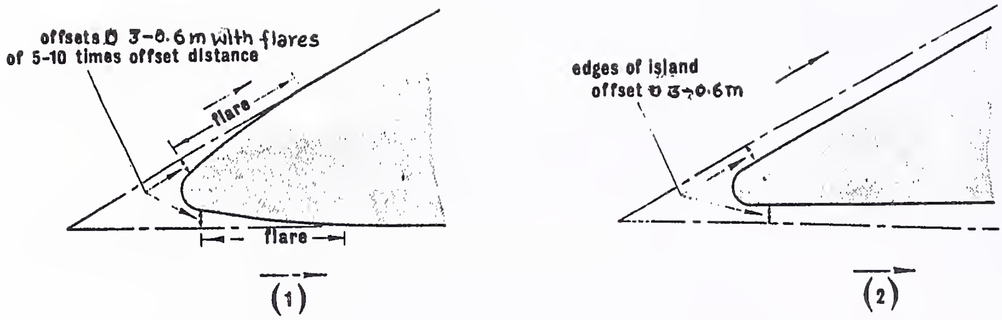


Fig. 4.21. Methods of Offsetting Approach Nose of Channelising Island

(b) It should be offset by about 1.5 m to 3 m from edge of main carriageway and suitably offset from approach centreline based on the track diagram of all turning movements (Fig. 4.24)

(c) It should be preceded by longitudinal and vertical profile which provides atleast the minimum acceptable sight distance.

(d) It should present a smooth, free flowing alignment into and out of the divided road.

(e) It should have excess width of pavement at the nose to create a "funnel effect".

(f) It should have sufficient length to warn drivers of the approaching intersection. A 3 sec. driving time of approach speed is sufficient.

(g) It should be sufficient width to permit use of bullet nose design with adequate right turn radii for vehicles and to permit placement of traffic control devices.

(h) It should not be less than 1.2 m wide and 6 m length. In special cases width can be reduced to 0.6 m.

The island shape is determined by wheel track diagram of vehicles using the road, the radii of left and right turns, island nose radii, approach pavement widths, general geometry of island and excess space to be covered by the island. Fig. 4.25 shows the correct method of shaping the centre traffic island while Fig. 4.26 shows the shapes of traffic island for different angles of turning. Fig. 4.27 shows the details of design of a divisional island. Fig. 4.28 shows the correct method of shaping a central island. The decision to provide an island or not should be based on examination of volume of traffic especially buses/trucks and the estimation of the probability of actual number of conflicts caused by right turning vehicles encroaching upon the stop position of vehicles on the intersecting road. In less volume rural roads or in residential streets, non-channelised intersection may be constructed as the few conflicts will not give economic justification for full fledged channelised intersection.

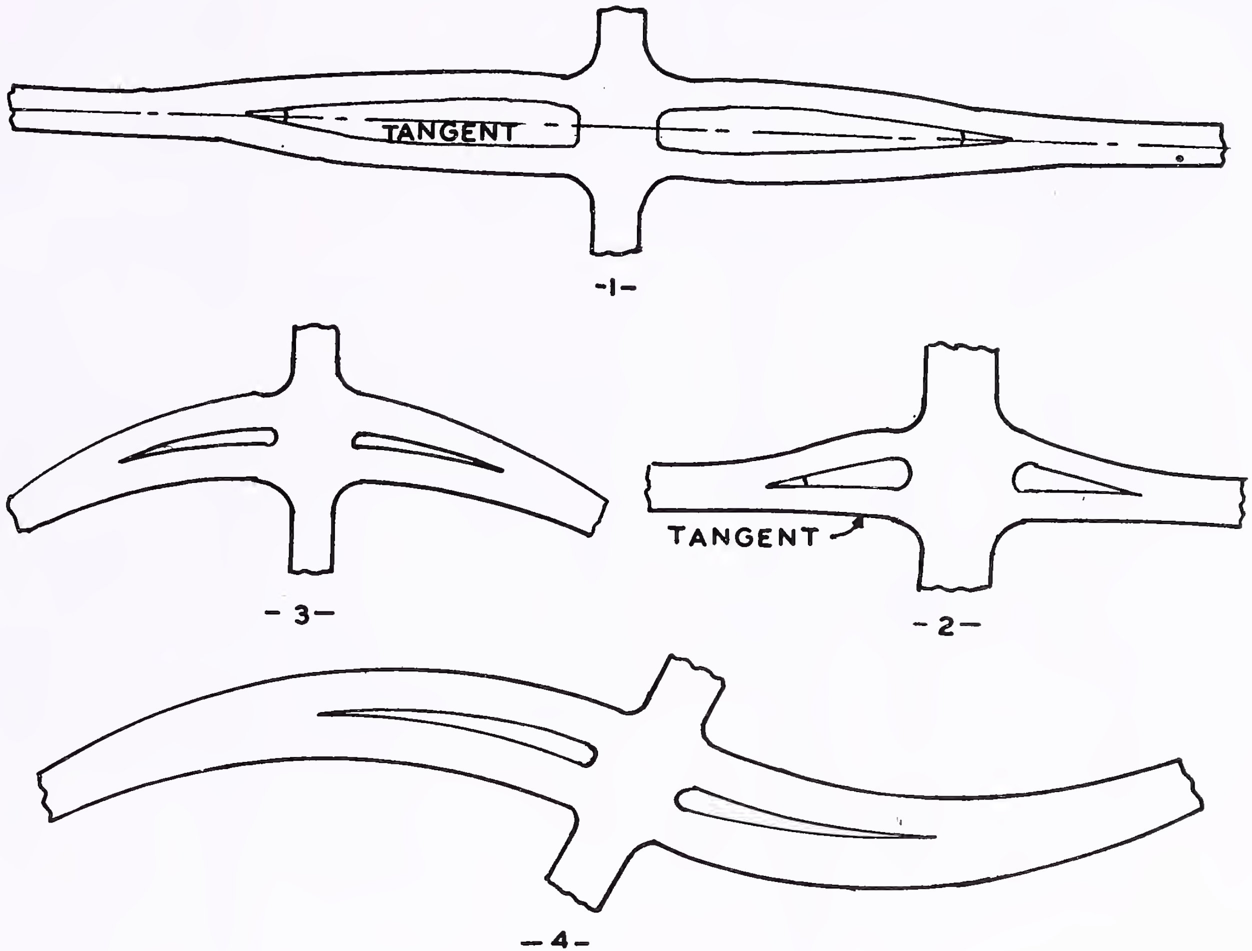
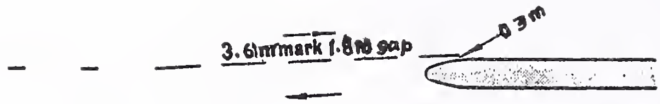


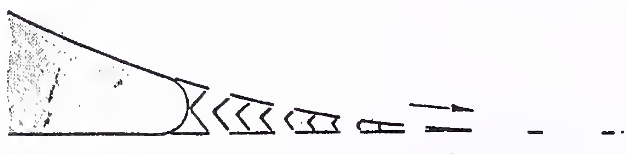
Fig. 4.22. Alignment for Providing of Divisional Islands at Intersections



(3) warning markings in advance of median island

(4) use of diagonal markings in advance of median island

(5) use of chevron markings when a traffic stream divides



(6) use of chevron markings where two traffic streams merge

(note: arrows indicate direction of traffic, not carriageway markings)

Fig. 4.23. Channelising Islands

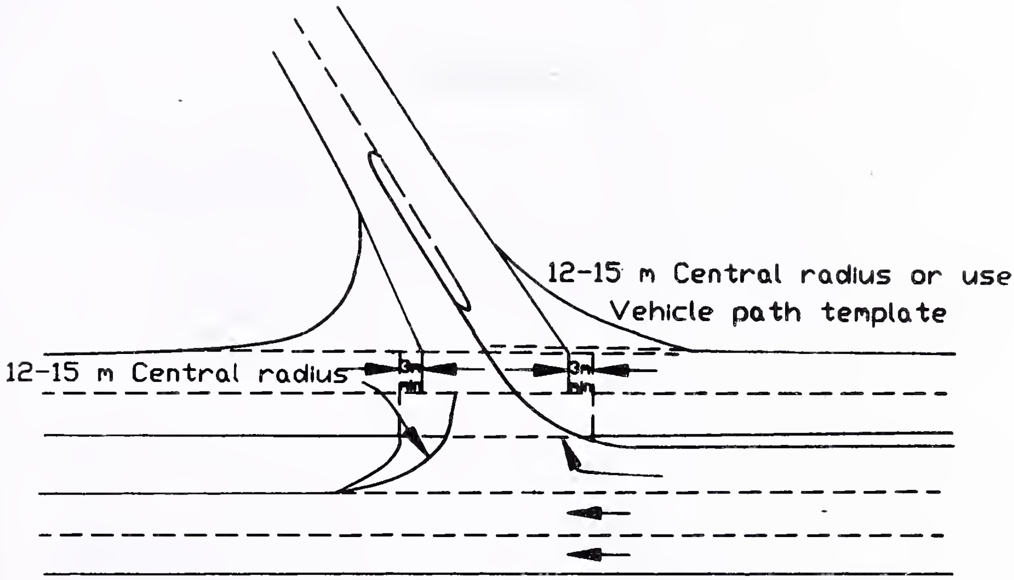


Fig. 4.24. Offset Details of Central Islands

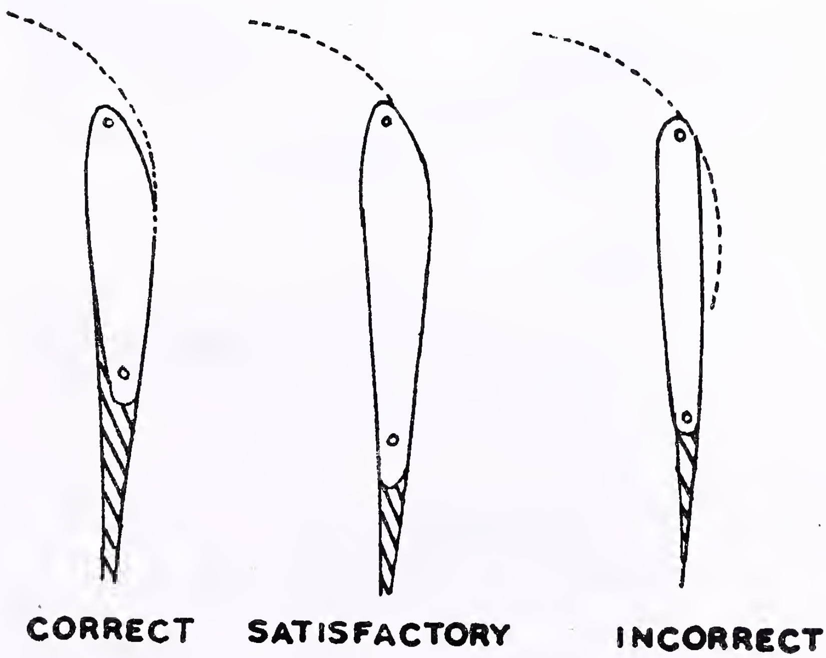


Fig. 4.25. Shaping the Traffic Island

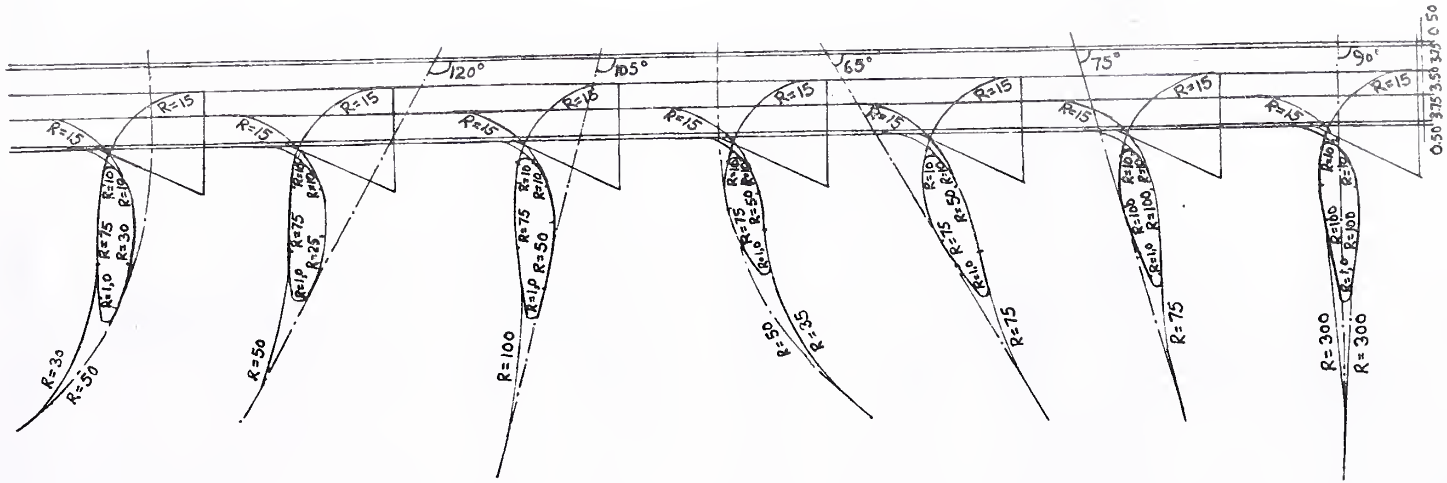


Fig. 4.26. Shaping of Traffic Island for Different Angles of Turning

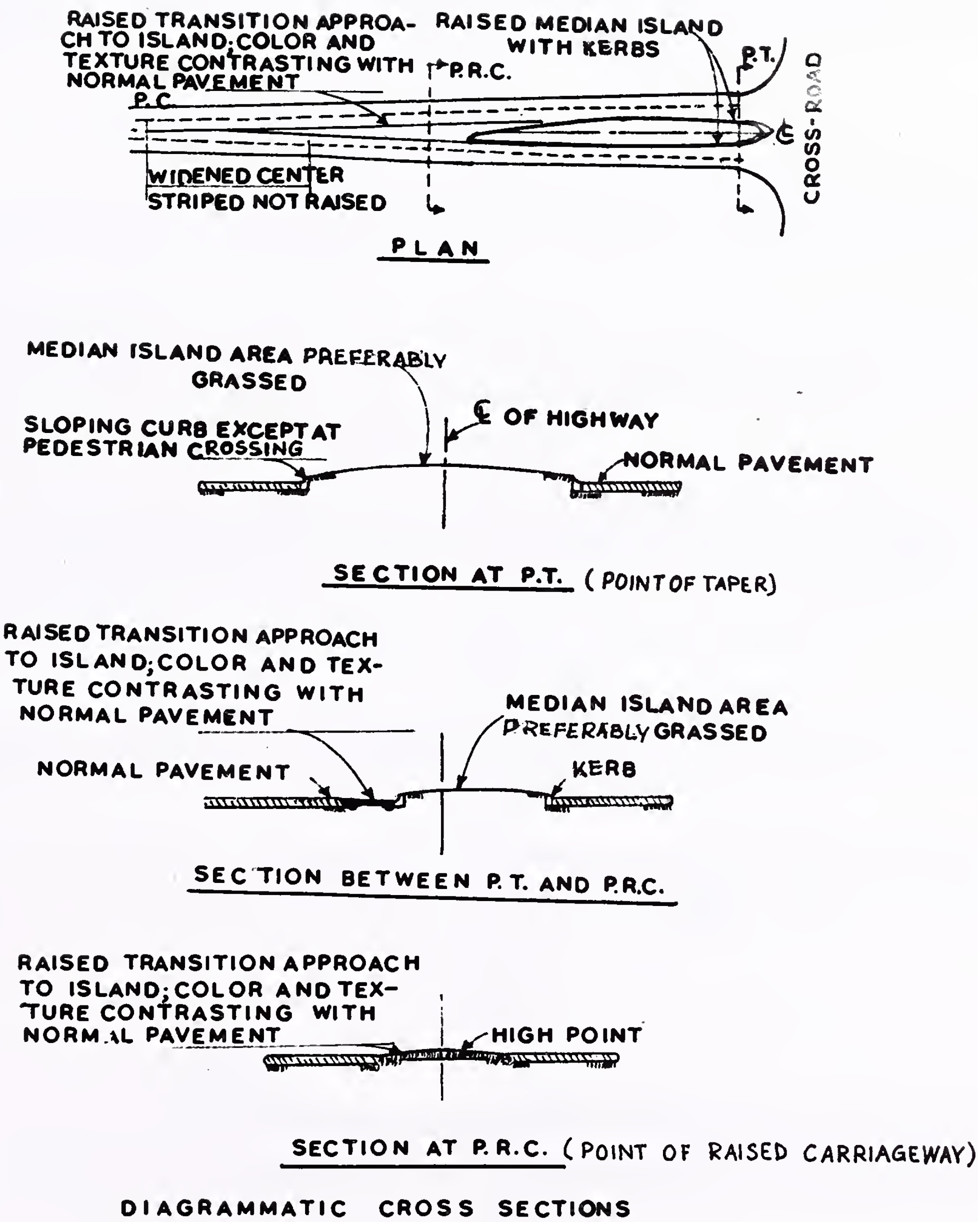
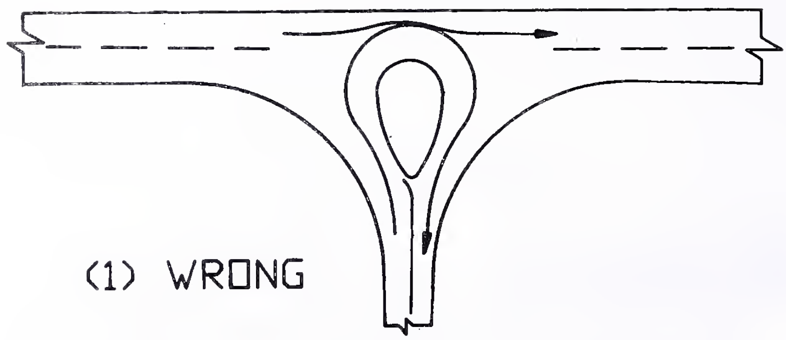


Fig. 4.27. Details of Divisional Island Design



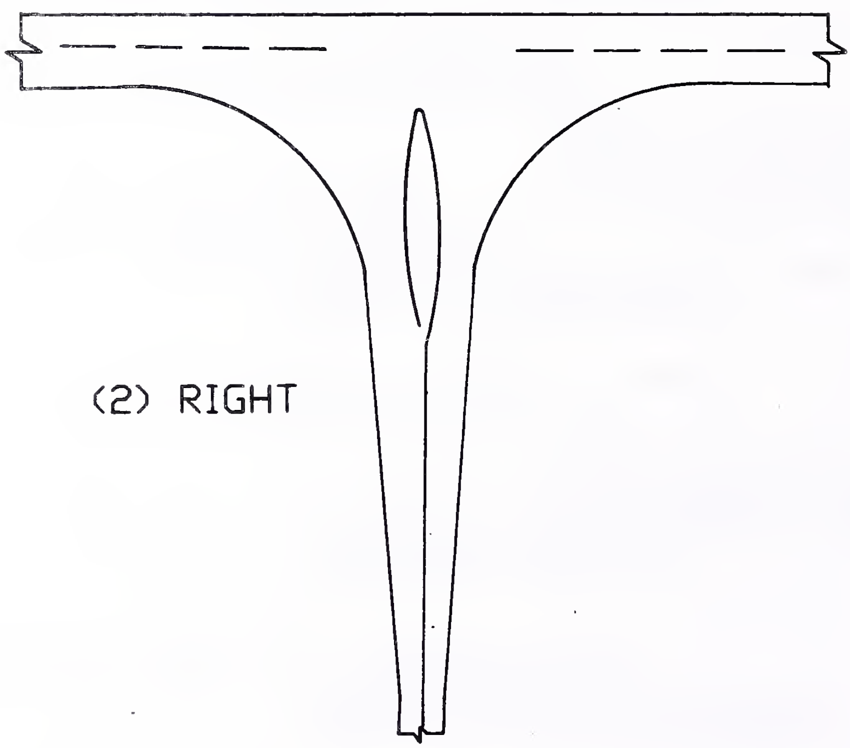


Fig. 4.28. Correct Method of Shaping a Central Island

4.11.4. **Pedestrian refuge island :** IRC : 70-1977 "Guidelines on Regulation and Control of Mixed Traffic in Urban Areas", provides general guidance on placement of pedestrian refuge island in urban areas. According to IRC : 103-1988, "Guidelines for Pedestrian Facilities", central refuges may be considered if the carriageway exceeds 4-lanes. The width of central refuge shall be 1.5m and above depending on the crossing pedestrian volume and space available. The refuge island should be provided with vertical kerb which should be suitably reflectorised and illuminated.

4.12. Considerations in Island Designs

4.12.1. An important point to be considered in island design is that its outline should be immediately obvious. It should be of easy flowing curved lines or straight lines parallel to the line of travel. Driver should not be confronted suddenly with unusable area and the island first reached should be indicated by gradually widening marking or a conspicously roughned strip that directs traffic to one side at easily traversed speeds. Multiple islands should be avoided and a few large islands should be preferred. It may be advisable to any temporary layouts of movable kerbs or send bags and observe traffic with several variation before designing and constructing a permanent island.

4.12.2. The use of kerbed island should be reserved for multi-lane highways or streets and for the more important intersections on two lane roads. In or near urban areas where speeds are low, drivers are accustomed to confined facilities and fixed lighting is possible, channelisation can be used freely. However, high unmountable kerbs with rigid angle-iron frames should not be used as they pose danger to life and property.

4.12.3. Where divisional islands are provided, approach roads should be so widened as to smoothly cater to all the directional traffic movement, see Figs. 4.5, 4.6 & 4.7. The alignment should be such that no undue effort in vehicle steering is required.

4.12.4. Drainage of highway after the introduction of island should be ensured. Large islands should be depressed inside to avoid draining across the pavement. Small islands can be mounted where pavement cross-slopes are outwards.

4.12.5. Island can be formed in three ways :

(1) raised island outlined by kerbs

(2) island delineated by pavement marking, buttons or raised rumble strips placed on all paved areas

(3) non-paved areas formed by the pavement edge supplemented by delineations or a mounded earth treatment beyond the pavement edge.

4.12.6. Typical layout of a T-intersection showing progressive use of islands to channelise traffic as dictated by increasing traffic is shown in Fig. 4.29. Fig. 4.30 shows typical details of channelises for a right hand splay junction.

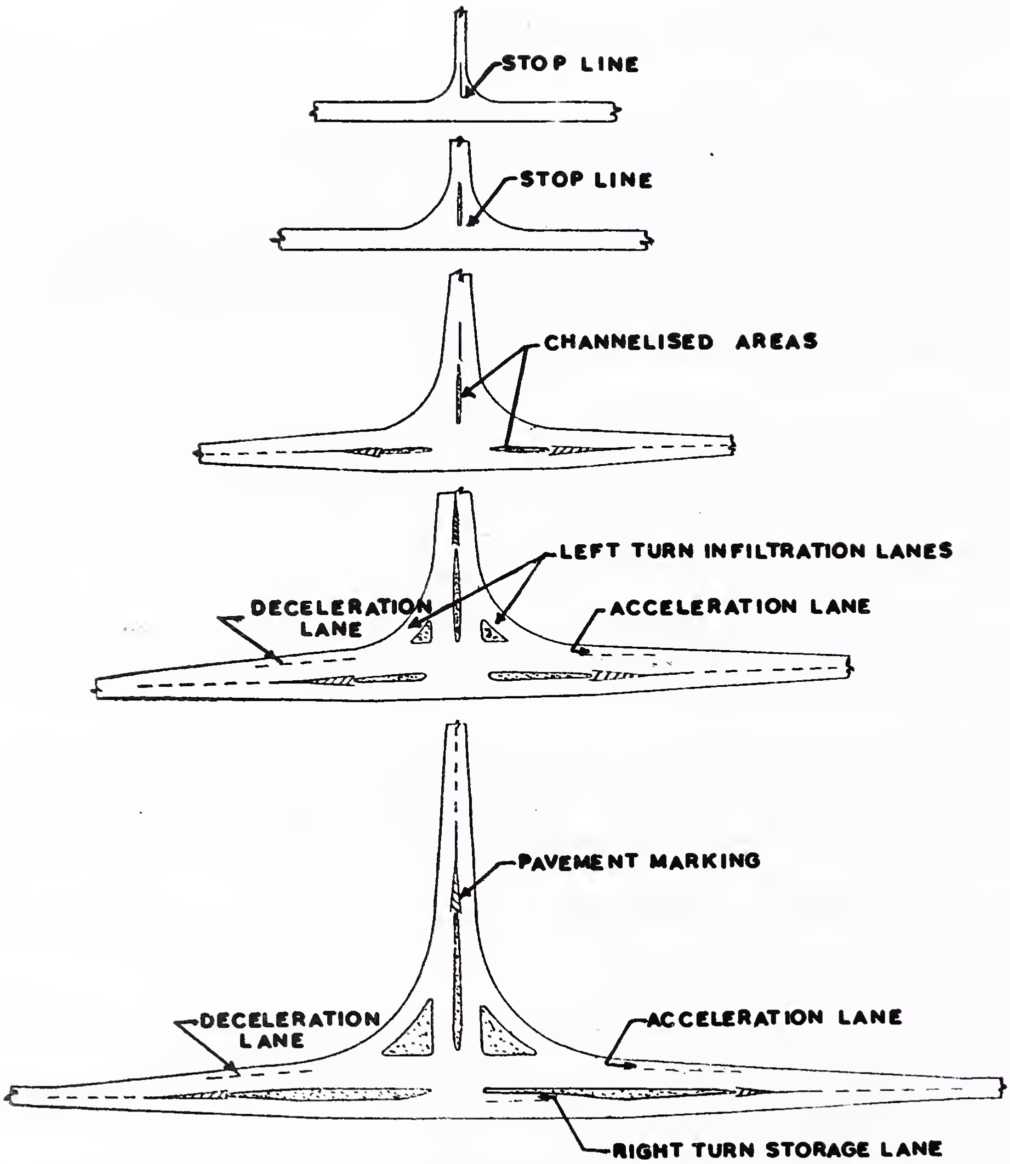


Fig. 4.29. Progressive Layouts of T—Intersections for use on main Highways.

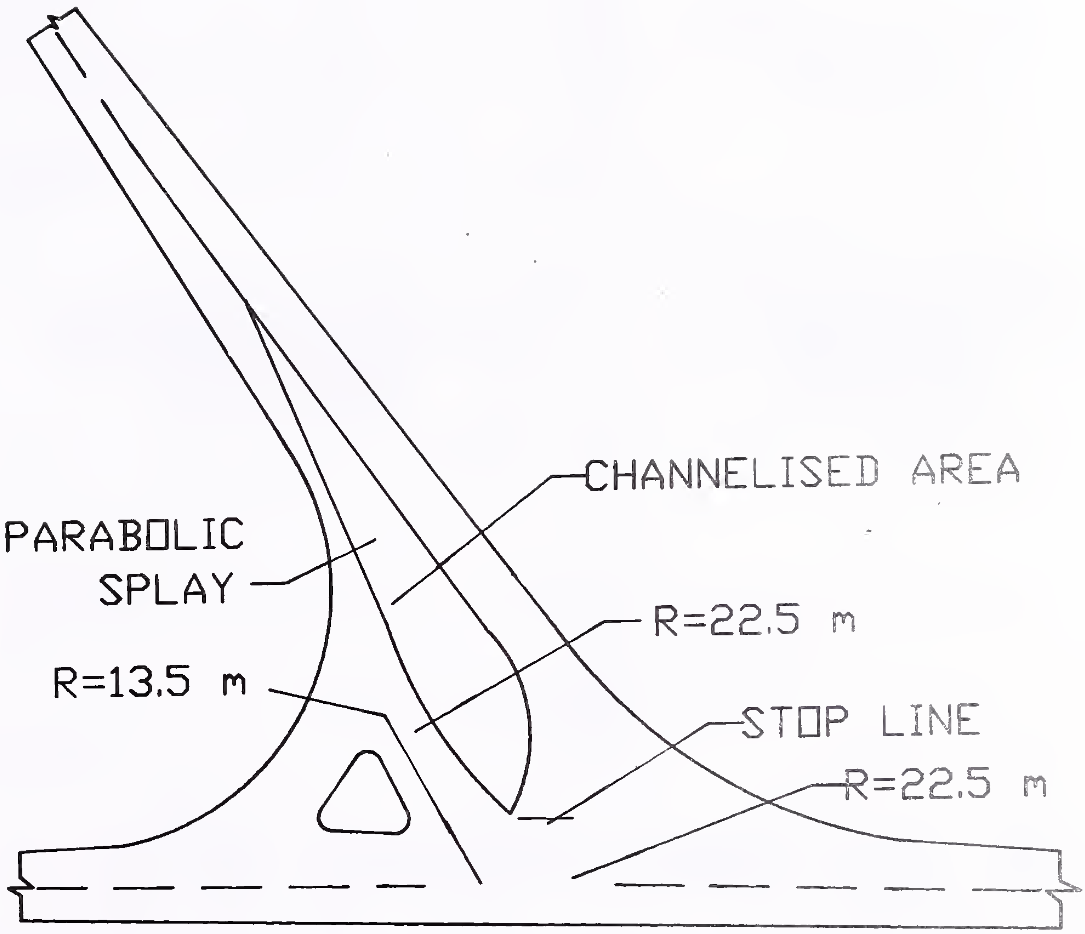


Fig. 4.30. Type Layout of a Right Hand Splay Intersection

4.12.7. A layout of four-leg intersection providing simultaneous right turns for a main road with a bullet nose is indicated in Fig. 4.31.

4.12.8. **Gap in central island of junctions :** To permit easy right turning of large vehicles, the gap in central reserve of junction should be extended to at least 3m beyond the assumed extension of kerb line of minor road and shape should be determined by 12 to 15m radius control circles tangential both in the centre line of minor road and the major road as shown in Fig. 4.24. To facilitate turning, the ends of the central island should be bullet nosed. Where a median right turn lane is to be accommodated, the design should be as shown in Fig. 4.32.

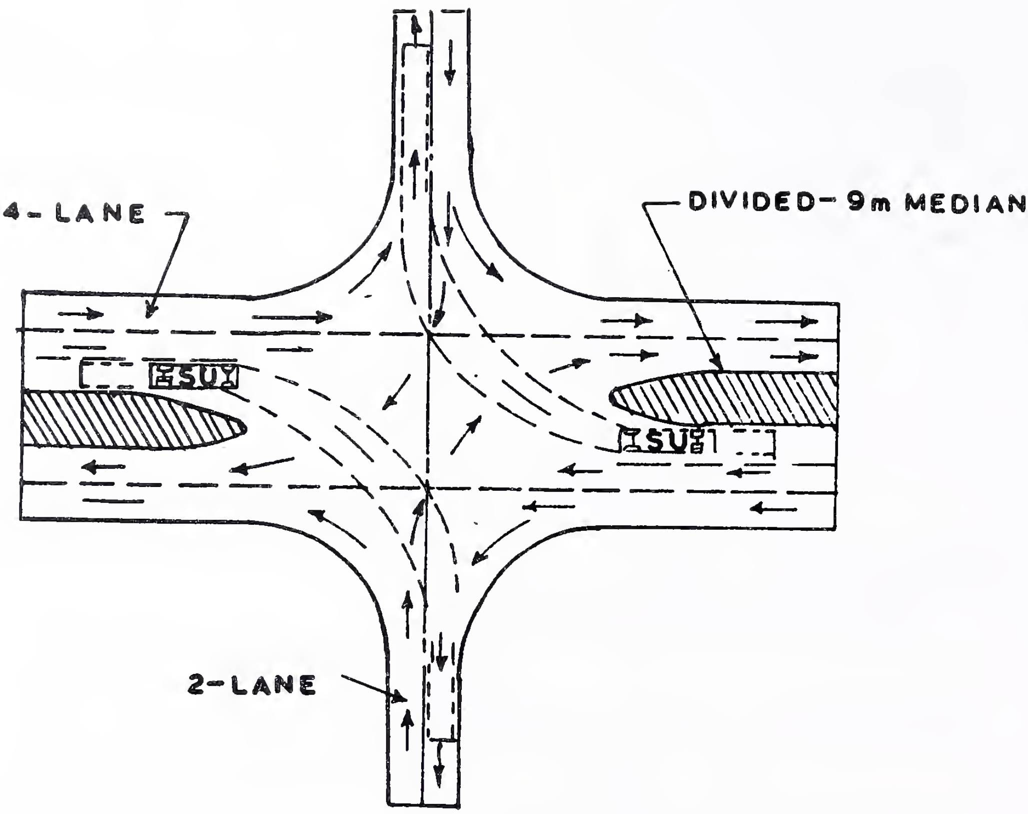


Fig. 4.31. Four-arm Intersection Providing Simultaneous Right Turns

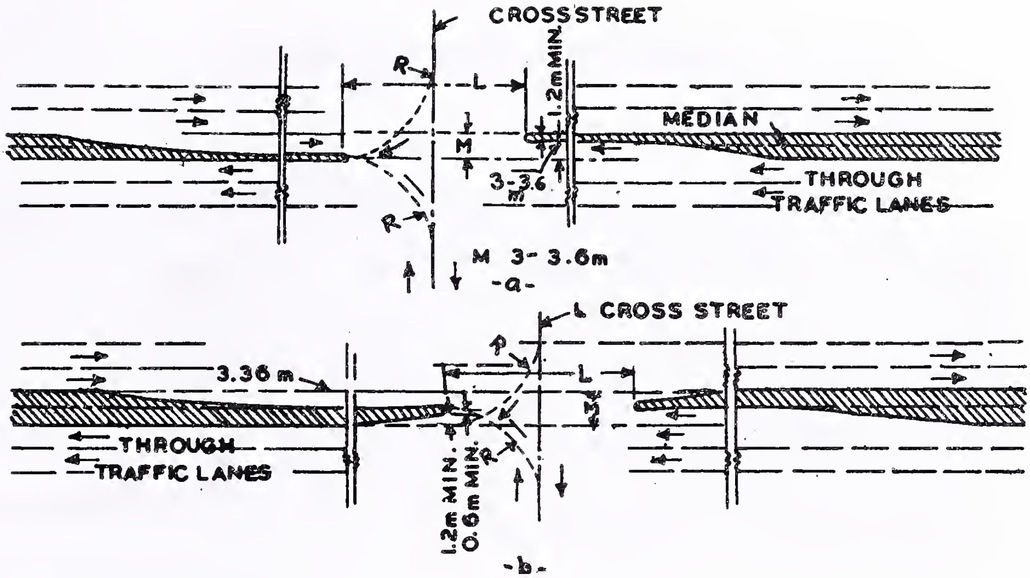
4.13. Kerb

It is desirable to provide kerbs at the intersections in urban and sub-urban area.

Kerbs may be barrier type, semi-barrier type or mountable type as shown in Fig. 4.33. Appropriate situations for use of each type are given in IRC : 86-1983 on "Geometric Design Standards for Urban Roads in Plains". The kerbs where used should be mountable type except for pedestrian refuge where they should be non-mountable.

4.14. Traffic Rotary

A traffic rotary is a specialised form of at-grade intersection where vehicles from the converging roads are forced to move around an island in one direction in an orderly and regimented manner and "weave" out of the rotary movement into their desired directions. The details of design of traffic rotaries are covered in IRC : 65-1976, "Recommended Practice for Traffic Rotaries."



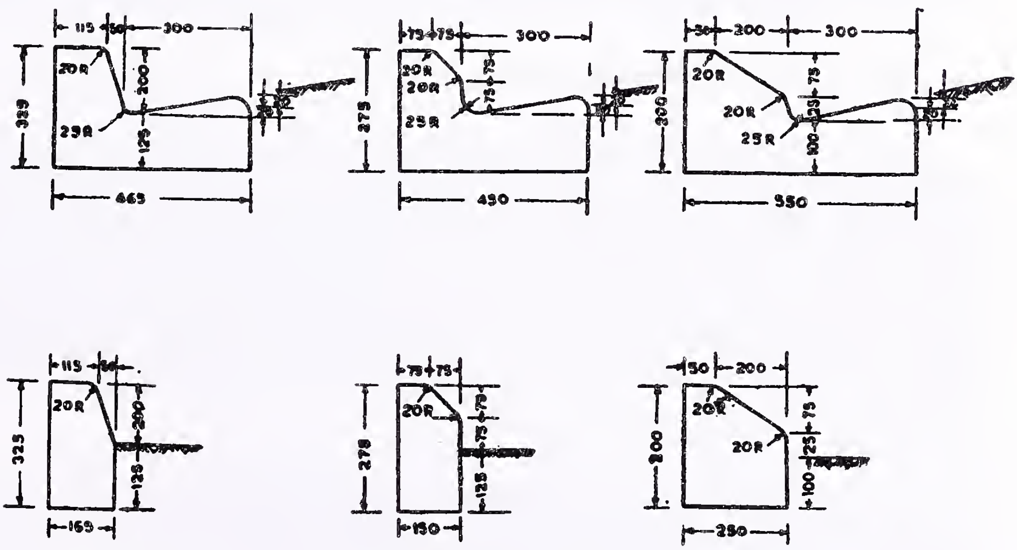
M = MEDIAN WIDTH

L = LENGTH OF MEDIAN OPENING

R = CONTROL RADIUS

PROVIDED IN ACCORDANCE WITH PARA 4.12.8

Fig. 4.32. Median Right Turn Design



(*a*) Barrier type

(*b*) Semi-barrier type

(*c*) Mountable type

All dimensions in millimetres

Fig. 4.33. Typical Kerb Sections

5. CAPACITY OF INTERSECTIONS

5.1. General

A road network (rural or urban) is generally classified into various categories according to the function the road performs. The type and forms of intersection should aim at providing the required capacity at level of service commensurate with the type of highway it connects. The capacity of most roads are influenced predominantly by the capacity and spacing of intersection, parking and unparking manoeuvres, percentage of buses and trucks, pedestrian movements, mix of traffic and geometric standards. It is, therefore, of paramount importance to assess the capacity of intersections and augment it through appropriate improvements so as to be compatible with the major highway capacity.

5.1.1. For the purpose of assessing capacity, the intersection can be classified under the following types :

(1) **Intersection with no-control :** Where the intersecting roads are of equal importance and traffic of neither road has precedence over the other.

(2) **Intersection with priority control :** There is theoretically no delay, occurring on the major road and vehicles on minor road are controlled by "GIVE WAY" or "STOP" signs.

(3) **Space-sharing intersections :** Like a rotary-where continuous weaving of traffic takes place.

(4) **Time-spacing intersections :** The right-of-way is transferred from one traffic stream to another in a sequence. Signalised and police-controlled intersections fall in this category.

5.2. Factors Affecting Capacity

5.2.1. The capacity of a road or intersection is the value of traffic flow, expressed in PCU's per hour, achieved under certain defined flow conditions. However,an intersection is a localised point in the road network where traffic streams conflict with one another and where the main operating characteristics is that of interrupted flow. Therefore the various streams within an intersection depend upon one another to a much greater degree than in the case of a road. Also, in case of an intersection the approach capacity is often more significant than that of a single stream. The capacity of the whole intersection can then be defined as the sum of all the flows traversing the intersection when one (or more in the case of traffic signals) approach becomes critical.

The capacity of uncontrolled intersection is, however, restricted as it depends on the frequency of gaps between vehicles in the main road flow which are of sufficient durations to permit vehicles from the side road to cross the main road. As, in general, the flow of traffic on a road approximates a random distribution, the volume of side road traffic which may cross selected volumes of main road traffic is based on minimum acceptable gaps and minimum vehicular spacing in the main and side roads.

5.2.2. The capacity of an intersection depends on the following factors :

(1) Physical and operating conditions :

- (a) Width of approach,
- (b) One way or two way operation,
- (c) Parking conditions/Ribbon development.

(2) Traffic Characteristics :

- (a) Turning Movements
- (b) Percentage of trucks & buses
- (c) Local factors at signalised intersection (ratio of the number of green phases that are fully utilised by traffic during peak hour to the total number of green phases available for that approach during the same period,
- (d) Peak hour factors (ratio between the number of vehicles counted during the peak hour and four times the number of vehicles counted during the highest 15 consecutive minute).
- (e) No. of Pedestrians and Cyclists crossing the intersection.
- (a) Location
- (b) Control Measures
  - (i) Traffic Signals and Signs
  - (ii) Marking of approach lanes

5.2.3. In India, very little data is available on actual estimates of capacities of intersection under mixed traffic conditions. The basic approach in evaluating capacity is to estimate traffic flows theoretically and then to modify the results based on actual information and identify factors which affect capacity. A further development in this direction is simulation modelling of traffic behaviour of intersection - so as to capture gap-acceptance behaviour and handways etc.

5.3. Capacity Determination

5.3.1. It may be noted that, due to the likely wide variations in local conditions, single level-of-service values and capacities cannot be presented even in situations having similar geometrics. A simple approach for capacity assessment of at-grade intersections is given in Appendix-II. For signalised intersection, a simple method based on U.K. practice is recommended in Chapter -7. Another approach based on Highway Capacity Manual of U.S.A. and research done in India is given in Appendix-III, which can be used after carefully understanding the procedure.

5.3.2. Intersection capacity should be equal to the capacity of the approach roads. The design capacity of 2-lane one way road may be taken to vary from 1400 to 2400 PCU's/hour (i.e. 700 to 1200 PCU's/ hour/lane). On a 2—lane two way road, capacity is 900 to 1500 PCUs/hour (i.e. 450 to 750 PCU's/hour/lane). In estimating the approach road capacity thought should also be given to road classification e.g. an arterial road is expected to have higher capacity than a sub-arterial or collector road.

5.3.3. In spite of all the difficulties inherent in estimating capacities of intersection, it can be confidently said that the following measures will improve the capacity of an intersection :

(1) On Major Roads

(a) Addition of acceleration and deceleration lane,
(b) Separation of two directions of traffic by a median, thus allowing crossing in two stages,
(c) Provision of separate storage lane for turning traffic.

(2) On Minor Road :

(a) Channelisation,
(b) Increasing number of lanes available, particularly the right and left turns.

5.3.4. **Capacity changes from the separation of conflicts :** The principle of separating traffic conflicts can be applied both to grade separated intersections and at- grade intersections. Separation of conflicts by the application of signal controls is more appropriate to urban or fringe urban areas. The capacity can also be augmented by fixing priority for the major road at intersections.

6. USE OF TRAFFIC CONTROL DEVICES AT INTERSECTIONS

6.1. Types of Devices

In intersection design, the possible use of traffic control devices and other road furniture should be considered. The common types of traffic control devices which are helpful in reducing accidents and improving flow conditions at intersections are road markings, road signs, signals, railing and flashing lights.

6.2. Road Markings

Carriageway markings within and in the neighbourhood of an intersection restrict vehicles from using areas other than those actually meant for them and thus ensure orderly movement of traffic. Depending on the actual intersection conditions, carriageway markings of intersections may consist of one or more of the followings :

(i) Centre line-solid or broken;
(ii) Solid centre lines preceded by broken centre lines on approaches to important intersections as an additional aid to channelise traffic ;
(iii) Centre line along with barrier lines;
(iv) Turn markings;
(v) Directions markings; and
(vi) Lane markings

Fig. 6.1 illustrates typical carriageway markings of road intersections. For more details, IRC 35 : 1970 (under revision) may be referred. In choosing the type of markings,

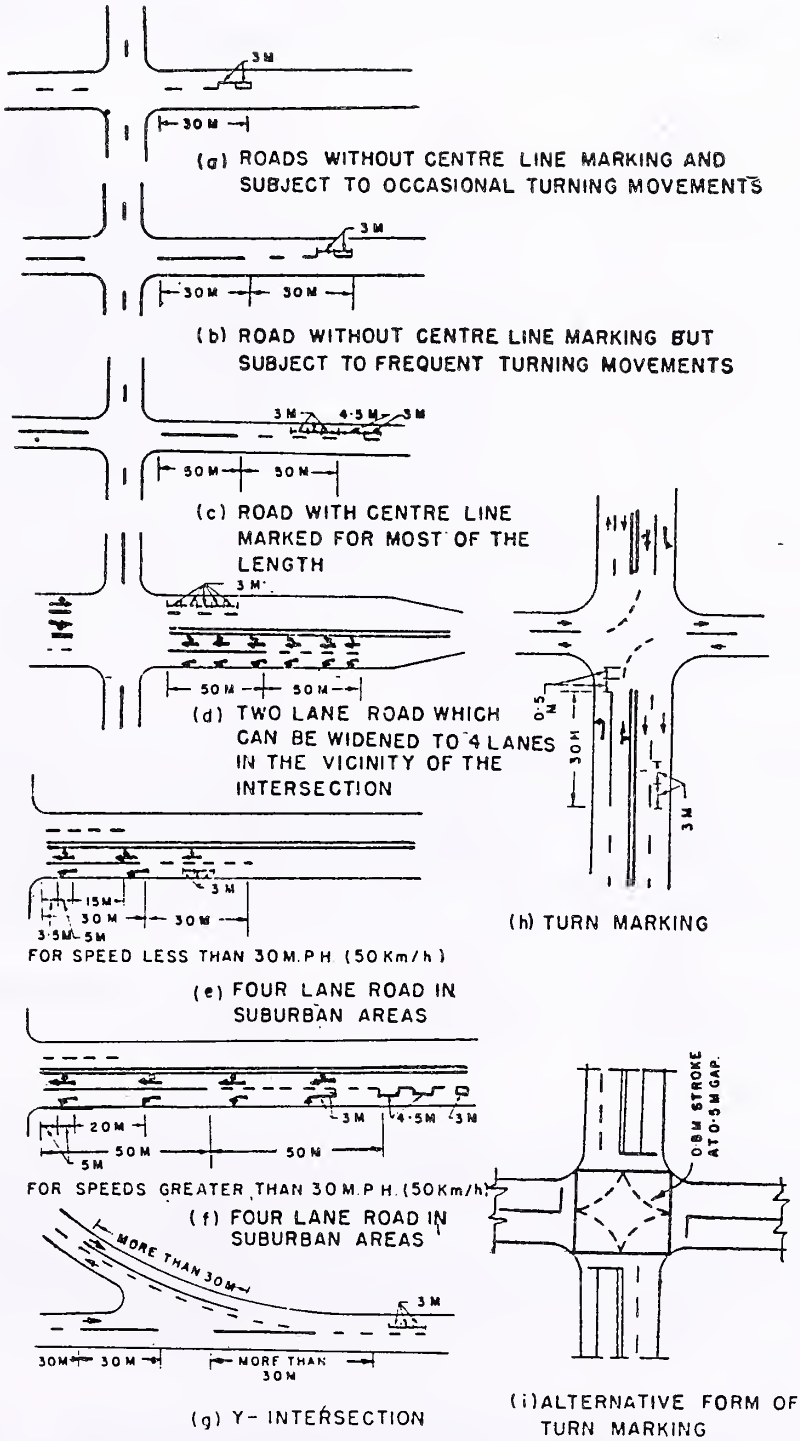


Fig. 6.1. Typical carriageway Markings at Road Intersections

discretion should be exercised by judiciously, considering the conditions at site where the lane markings are to be provided, care must be taken to ensure continuity of lanes across the intersection.

6.3. Signs and Signals

To properly guide and control the traffic approaching junctions, traffic signs and signals are installed. IRC : 67-1977 "Code of Practice for Road Signs" provides a more detailed description of various traffic signs and their applications. Design and provisions of traffic signals has been dealt separately under section 7 of these guidelines. While providing traffic signs at intersection, due care must be taken to ensure the effectiveness of the sign posted. It should always be noted that too many signs with inadequate spacing may become totally ineffective and at times result in confusion and accidents. Some of the traffic signs which are useful in guiding/regulating the traffic at intersections are shown in Fig. 6.2. For other details on their sizes and colour codes, IRC : 67-1977 may be referred. The posting of traffic signs must be done with adequate care so that they perform their intended function most effectively, in as much as the sign must be posted ahead of the spot to which it refers. When more than one sign is to be posted, they should be adequately separated in space so as to be seen one at a time and convey the message with complete effect. Signs with reflective properties (preferably retro-reflective type) must be used so as to meet the requirements of night traffic.

While posting the signs, adequate care should be taken so as to avoid the chance of their causing obstruction to pedestrian and vehicular traffic. In urban areas the lowest edge of any traffic sign should not be lower than 2.1 m from the pavement when posted on foot-paths/side walks. In rural areas the clear height of sign from the edge of the pavement should be 1.5 m. The nearest edge of the sign should be at least 1.2 m away from the edge of the carriageway on rural roads. When posted on raised foot-paths the same should be away by at least 30 cm from the edge of the kerb.

A few typical examples of posting traffic signs at urban and rural intersections are illustrated in Figs. 6.3 to 6.6.

6.4. Reflectors

A large percentage of accidents at intersections occur at night due to poor visibility and hazards like traffic islands, median openings or other objects close to the carriageway. At such locations, use of reflector units of suitable dimensions can improve safety considerably. A typical unit suitable for use on island consists of three circular reflective units mounted on triangular plate as per IRC : 79-1981. In urban or sub-urban areas, safety is best improved by illuminating the entire intersection area adequately.

6.5. Railings

The requirements of railings at intersection have been dealt in IRC : 103-1988 "Guidelines for Pedestrians Facilities" which may be referred to for more details.

Fig. 6.2. Signs Used at Intersections

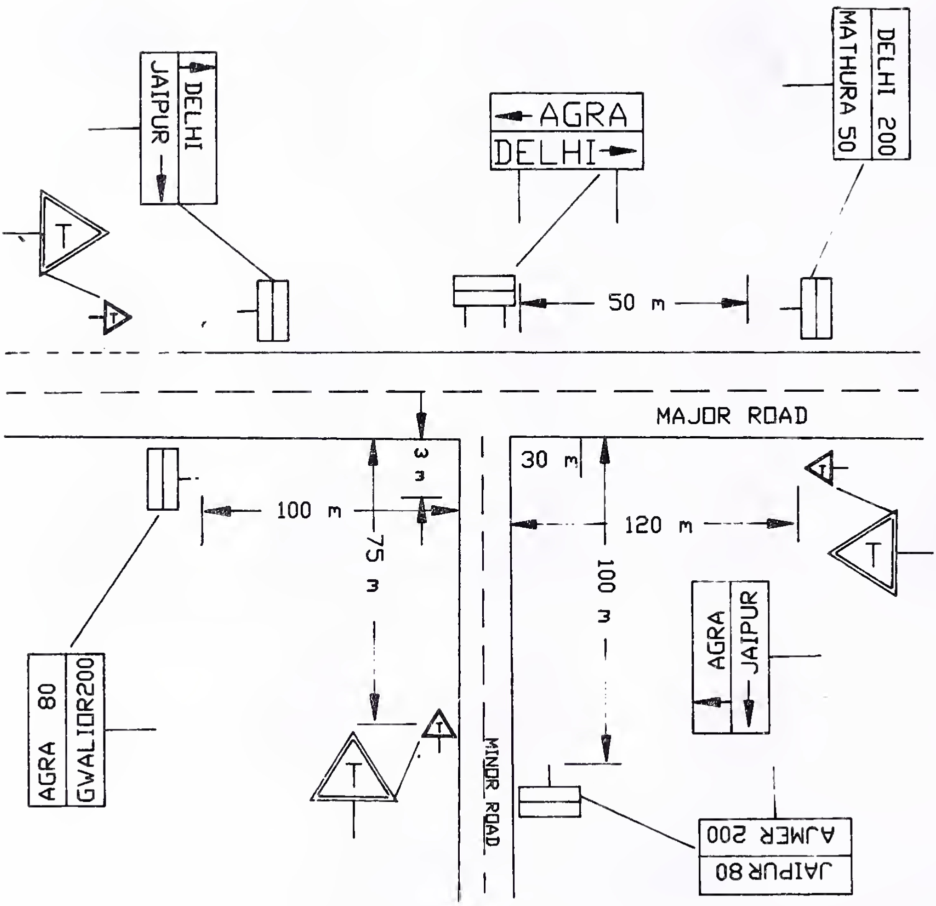


Fig. 6.3. Typical Sign Postings at Rural T-Intersection

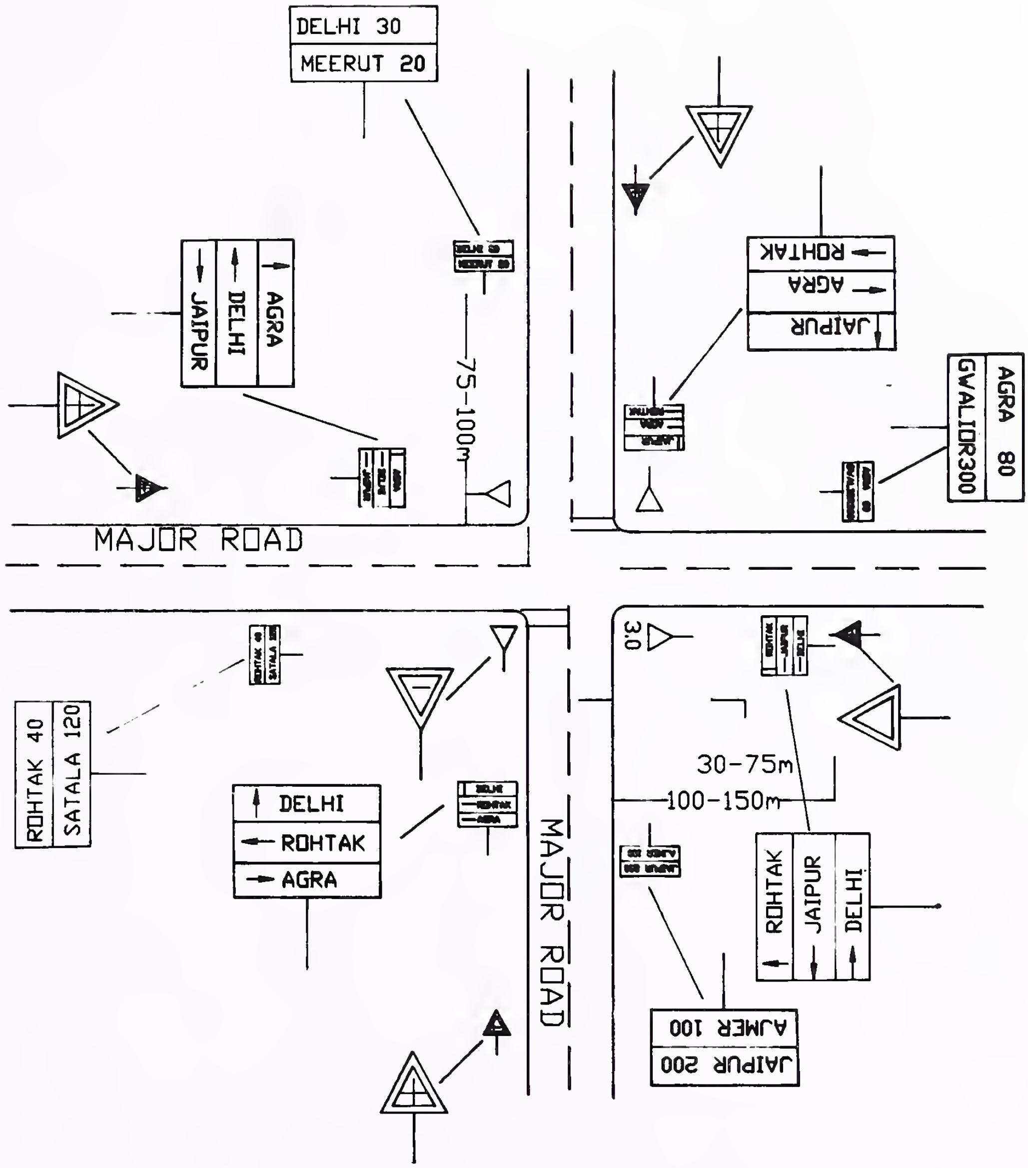


Fig. 6.4. Typical Sign Posting at Rural Four Arm Intersection (Four-Arm Intersections)

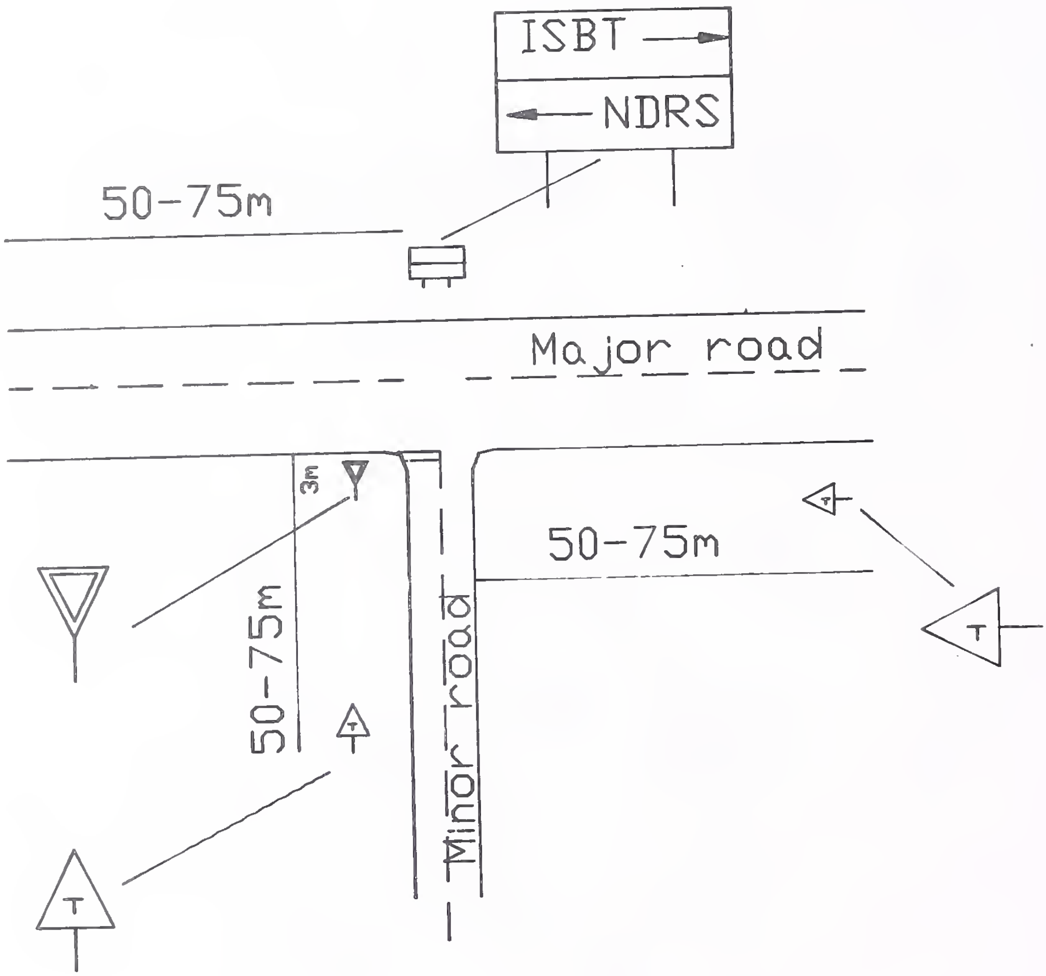


Fig. 6.5. Typical Sign Posting at an Urban T-Intersection

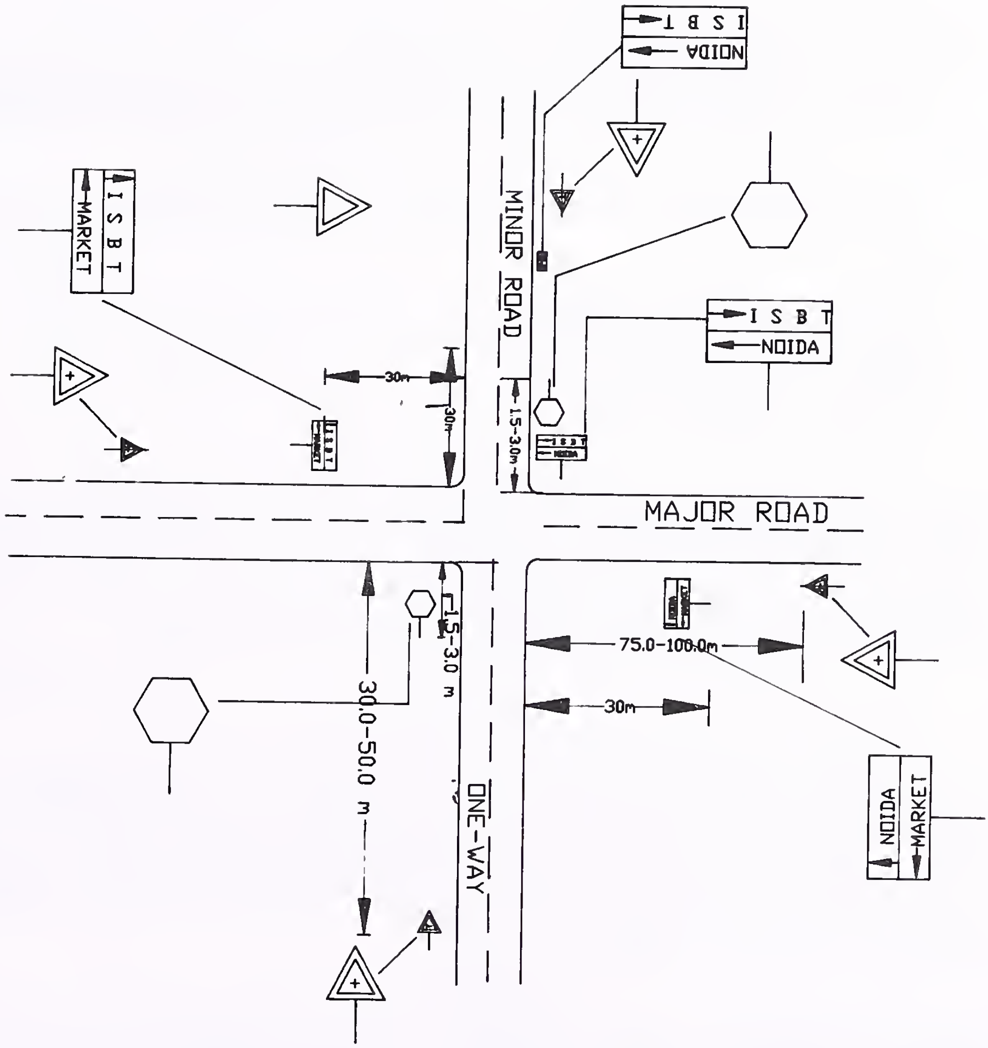


Fig. 6.6. Typical Sign Posting at an Urban Four-Armed Intersection

7. SIGNAL CONTROLLED INTERSECTION

7.1. The primary aims of signal control at an intersection are :

(i) To reduce traffic conflicts and delay,
(ii) To reduce accidents,
(iii) To economise on police time.

7.2. Warrants for Traffic Signal Installation

Before the traffic design process can commence, it must be determined whether or not the volume of traffic entering the intersection warrants signalisation.

The need for a traffic control signal at any intersection must be evaluated in relation to the following warrants :

Warrant 1 - Minimum Vehicular Volume,
Warrant 2 - Interruption of continuous traffic,
Warrant 3 - Minimum pedestrian volume,
Warrant 4 - Accident experience,
Warrant 5 - Combination of Warrants.

The traffic control signals should not be installed unless one or more signal warrants as discussed in Section III of IRC : 93-1985 "Guidelines on Design and Installation of Road Traffic Signals" are met. The satisfaction of a warrant, however, does not always completely justify signal installation. Engineering judgement must be exercised to ensure that the signal could not increase the hazards or cause unnecessary delay.

7.3. Traffic Survey

The initial step may consist of a quick review of existing volume data and accident records. A field visit to the location is necessary to observe the existing conditions and intersection geometry. At those locations where further study appears justified, the next step is to schedule and plan the engineering studies appropriate to the particular location which will indicate whether more detailed data is required for signal design and plan preparation if a signal is warranted.

A comprehensive investigation of traffic conditions and fiscal characteristics of the location is essential to determine the necessity for a signal installation and to furnish necessary data for the appropriate design and operation of signal that is warranted. The advance traffic and engineering data required have been discussed in Section III-1 of IRC: 93-1985.

7.4. Geometric Design

Following parameters are to be considered.

7.4.1. **Selecting approach widths :** Because signals permit traffic movement from any approach for only a portion of the time it is sometimes necessary for the intersection approaches, where queuing takes place to be wider than the roads which feed these approaches in order to pass the required flow (Figs. 7.1, 7.2 and 7.3 refers). In selecting approach widths one criteria which can be used is the minimisation of the area occupied by the intersection.

For a normal two phase cross roads, approach width (W_1 and W_2 as in Fig. 7.1) should be proportionate to the square roots of the ratio of flows, q_1 and q_2 respectively. The green times (g_1, g_2) and length of widened road (d_1, d_2) (as in Fig. 7.1) should be in the same ratio as the width W_1 & W_2 respectively.

$$\frac{W_1}{W_2} = \frac{g_1}{g_2} = \frac{d_1}{d_2} = \sqrt{\frac{q_1}{q_2}}$$

Where

q_1, q_2 = maximum flow in PCU in phases 1 & 2 respectively.

q = flow i.e. the average number of vehicles passing a given point on the road in the same direction per unit time.

Phase = State of signals during which a particular stream or group of streams receives right of way.

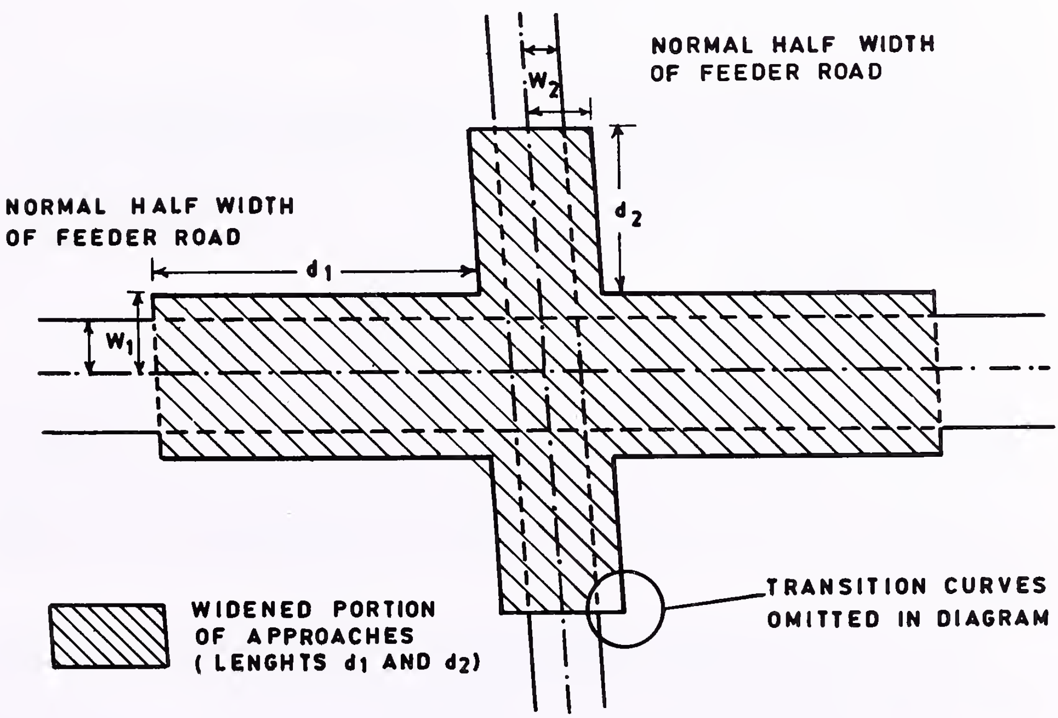


Fig. 7.1. Simplified Diagram of Widened Approaches at Signal Controlled Intersection

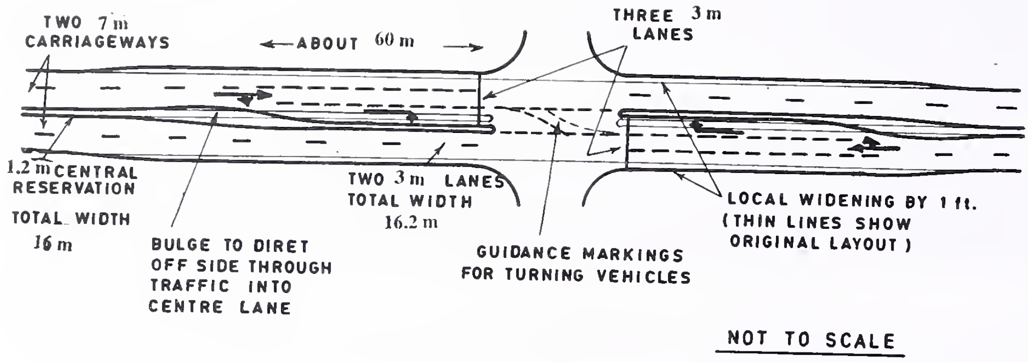


Fig. 7.2. Suggested Layout for Offset Central Reserve

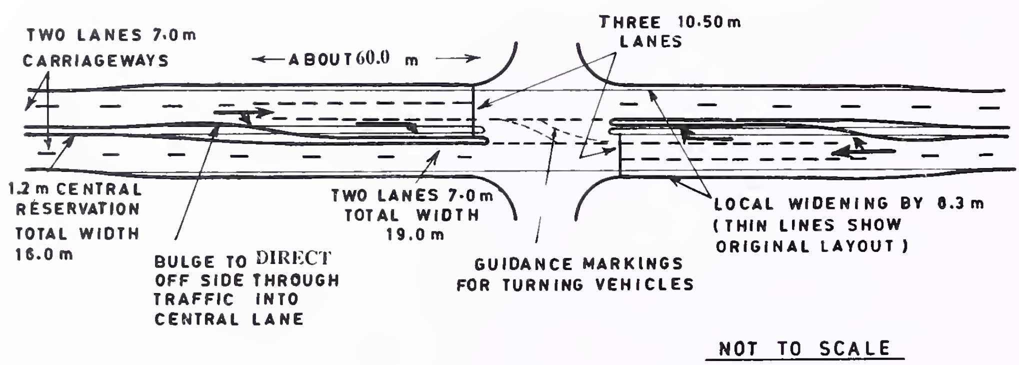


Fig. 7.3. Offset Central Reserve with Improved Visibility for Right Turning Vehicles

Thus a major road carrying four times as much traffic as its minor cross road should have approaches which are twice as wide as the minor approaches and should have green times twice as long.

For T-intersections with two phase control the ratios of width, green times and lengths widened should be :

$$\frac{W_1}{W_2} = \sqrt{\frac{q_1}{2q_2}} \text{ and} \qquad \text{... (2)}$$

$$\frac{g_1}{g_2} = \frac{d_1}{d_2} = \sqrt{\frac{2q_1}{q_2}} \qquad \text{... (3)}$$

where the suffix 2 refers to the minor cross road. Thus a major road through a T-intersection carrying four times as much traffic as the minor cross road should have a width 1.4 times than that of cross road, a length of widening 2.8 times and a green period 2.8 times as long as that of a minor cross road.

7.4.2. The flow can be determined by a census of all traffic using the intersection over a day including all peak periods. Calculations should cover morning, evening and other peaks to determine the predominant flows at different times of the day. The width required for any approach not considered may be determined from the green time allotted to its phase. In deciding the values of q_1 and q_2 for substitution in the above formula, right turning vehicles if few in number should be taken as equivalent to 1.75 of straight ahead vehicles. If right turning volume is large enough to require special right turn lanes this should be substracted from the flow q_1 and q_2 before determining the ratios of remaining approach widths.

Where uniform widening of an approach is prevented by the presence of existing buildings, it may only be possible to flare the approach. On dual carriageway roads extra width may sometimes be obtained by reducing the width of the central reserve.

7.4.3. **Selection of lane widths :** It is normal to have lanes 3 m wide at new or improved intersections, through occasionally 2.8 m lanes may have to be accepted at existing intersections because of site constraints. A lane width of 3.4 m or 3.6 m may be warranted where traffic includes a larger proportion of commercial vehicles. Where there is a high proportion of bicycles it may be beneficial to have a wider (upto 4.0 m) near side lane.

Approach widths may need to be adjusted to provide appropriate integral number of lanes for straight through and turning traffic, having regard to the overall width available. Where the approach width is less than 5.5 m no lane markings should be provided; for widths between 5.5 m and 7.5 m the approach should be marked two lanes; for widths between 7.5 m and 9 m the approach should be marked as three lanes. All lane width need not be the same and should be chosen with regard to the volume and type of traffic using each lane.

7.4.4. **Layout for right-turning vehicles :** Opposing right-turnings traffic can turn either on the off side of each other or on the near side. If the near side method of turning right is used there may be advantages in off setting the central line or central reserve, so that more space is available to traffic approaching the intersection than to traffic leaving it. For exceptionally high flow of right turning vehicles, it is sometimes necessary to provide more than one lane for them with separate phase. A separate phase should be provided if there are many right turners from both approaches using near side method of turning. In some cases it may be desirable to place the opposing right turners opposite each other as in Fig. 7.2. Layout suggested in Fig. 7.3 improves visibility for right turners and facilitates the signalling of the right turners if they move on a separate phase.

If right turning traffic from one approach is heavy, the green period of this approach may continue after the end of green period of the opposing arm sharing the same phase (Fig. 7.4).

The layout shown in Fig. 7.5 includes a pedestrian crossing which is a usual nature of intersection where there are appreciable number of pedestrians. A pedestrian refuge is usually placed at or near the centre of a single carriageway if the widths available to traffic in the two directions are greater than 4.0 m.

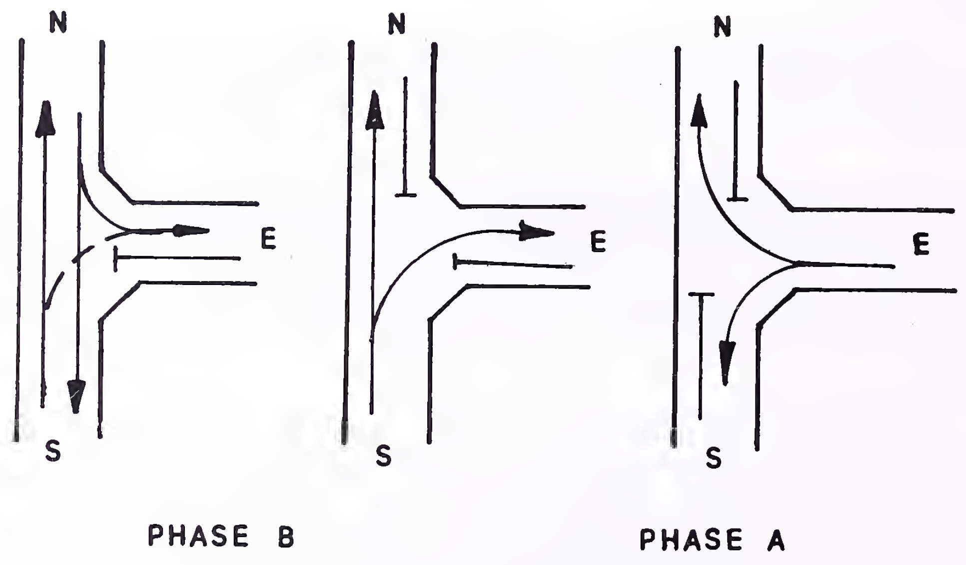


Fig. 7.4. Early Cut-off

7.5. Signal Design

Determination of cycle lengths and green periods in signal phasing alongwith typical design of signal timings are discussed in Section II(22) of IRC : 93-1985.

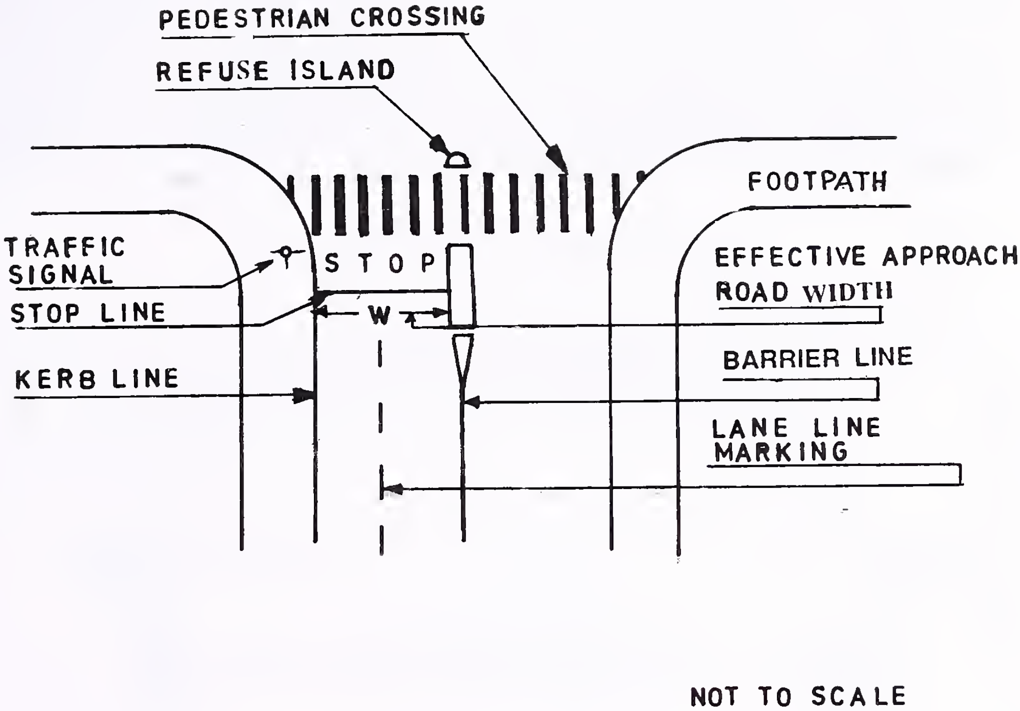


Fig. 7.5. Typical Intersection Approach Road Showing Pedestrian Crossing

7.6. Intersection Capacity

The amount of traffic that can pass through a signal control intersection from a given approach depends on the green time available to the traffic and on the maximum flow of vehicles past the stop time during the green periods. The capacity and service volume that a signalised intersection can handle depends upon geometrics, signal operation and traffic factors. The capacity of a signalised intersection can be determined from the expression :

Capacity = (g x s)/c vehicles per hr.

Where g = effective—green time in seconds

s = the saturation flow (vehicle per hr.)

c = cycle time in seconds.

These parameters can be determined as follows :

7.6.1. **Saturation flow :** The saturation flow is the flow which would be obtained if there is a continuous queue of vehicles and they were given 100 per cent green time. It is generally expressed in vehicles per hour of green time. When the green period commences, vehicles take some time to start and accelerate to normal running speed but after a few seconds the queue discharges at a more or less constant rate which is called saturation flow. This concept is explained in Fig. 7.6 wherein the effective green time has been explained through which the flow is assumed to take place at saturation rate and a lost time during which no flow takes place. The curve in Fig. 7.6 can be replaced by a rectangle of equal area where the height of the rectangle is equal to the saturation flow. Saturation flow can be estimated by the following methods :

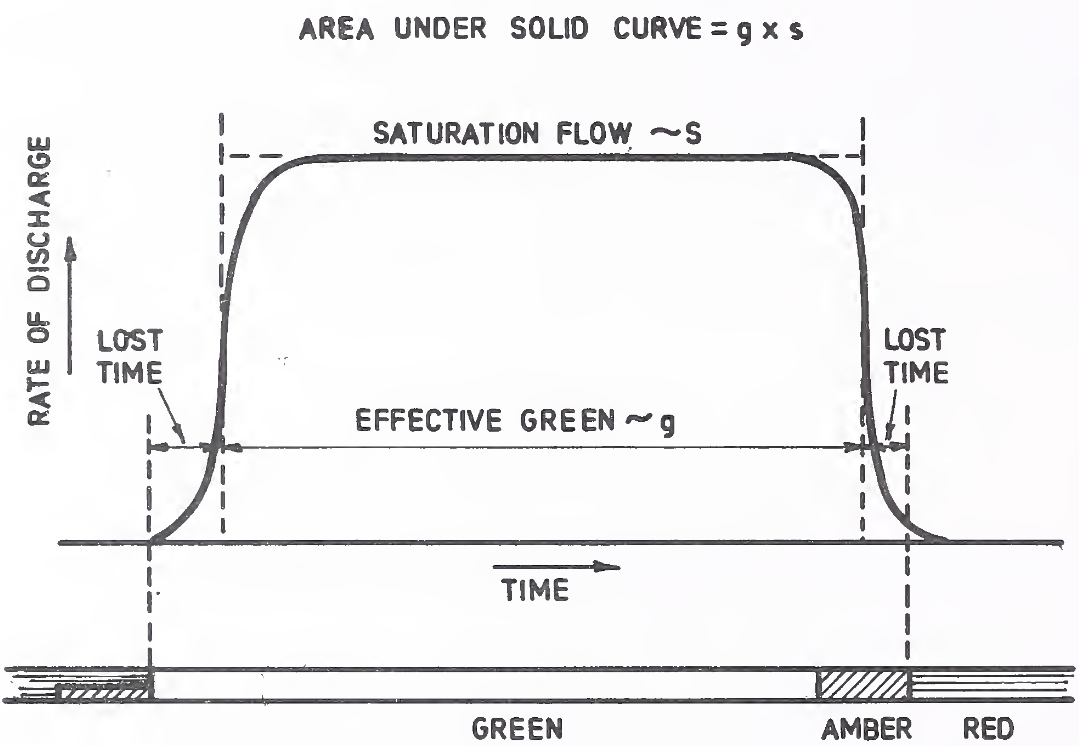


Fig. 7.6. Saturation Flow and Lost Time

7.6.1.1. **Approach width :** The saturation flow(s) expressed in terms of Passenger Car Units (PCUs) per hr. and with no parked vehicles present is given by

s = 525 (w) PCUs per hr.

where w = width of approach road in m. measured from kerb to the inside of the central median or centre line of the approach whichever is nearer.

This expression is valid for widths from 5.5 m to 18 m. For a lesser width the values may be obtained from the table given on next page :

Width w in metre	3.0	3.5	4.0	4.5	5.0	5.5
Saturation flow(s) (PCUs) per hr.	1850	1890	1950	2250	2550	2990

7.6.1.2. **Effect of Gradients :** For each 1 per cent of uphill gradient, the saturation flow decreases by 3 per cent and for each 1 per cent of downhill gradient, saturation flow increases by 3 per cent. The gradient shall be the average slope between the stop line and a point on the approach 60 m before it.

7.6.1.3. **Effect of right turning traffic :** If the right turning movements from opposite directions cause the intersection to lock, then the capacity of the intersection cannot be easily assessed. Under non-locking conditions, the effects of right turning traffic depend on whether or not conflicting traffic, moves on the same phase and on whether or not the right turning traffic is given exclusive lanes. There are four possibilities.

(i) No opposing flow, no exclusive right turns : The saturation flow can be obtained as from 7.6.1.1.

(ii) No opposing flow, exclusive right-turning lanes : The saturation flow(s) depends on the radius of curvatures (r) and is given by :

$$s = \frac{1800}{1 + \frac{1.52}{r}} \quad \text{PCUs/m for single file streams}$$

and $$s = \frac{3000}{1 + \frac{1.52}{r}} \quad \text{PCUs/m for double file streams}$$

where r is radius of curvature in metres of the right turning steam through a right angle.

(iii) Opposing flow, no exculsive right turning lanes : The maximum number of right turning vehicles per cycle that can take advantage of gaps in the opposite stream can be determined from the following equations.

$$n_r = s_r \frac{(g \times s) - (g \times c)}{s - g}$$

Where s_r = right turning saturation flow

q = flow in opposing arm

s = saturation flow for opposing arm

g = green time

c = cycle time

(iv) Opposing flow, exclusive right turning lanes : There should be no delay to the straight ahead traffic using the same approach as the right turners, but there will be an effect on the cross phase and this should be calculated using the same method as in (iii) above.

7.6.1.4. **Effect of mixed traffic condition :** The saturation flow gets reduced due to presence of slow moving vehicles in the stream as the turning vehicles shall have to find the suitable gap thereby affecting the average delay in the cycle.

7.6.2. **Lost time :** It is the time during which no flow takes place. It may be

(i) The total lost time per cycle (L)—The sum of lost time in each phase and this period when all signals show red or red-amber. It can be expressed by,

$$L = nl + R$$

Where n = number of phases
l = average lost time per phase (excluding any all red periods or sequent ambers)
R = all red period time during each cycle where all signals display red or red with amber.

(ii) the lost time for a single phase (l)—The amount of time in a cycle which is effectively lost to traffic movement in phase because of starting delays and the falling of the discharge rate during amber period. No flow takes place in lost time as would be seen in Fig. 7.6.

For the average signals cycle the lost time caused by starting delays and reduced flow during the amber period amounts to about 2 seconds per phase.

7.6.3. **Cycle time :** The total time period required for one complete sequence of signal indications is the cycle time. The cycle time with minimum delay could be represented by.

$$Co = (1.5L + 5)/(1 - Y) \text{ seconds}$$

where L = total lost time per cycle in seconds, generally taken as the sum of total amber and all red clearance per cycle in seconds. $Y = y_1 + y_2 + ... y_n$ where $y_1, y_2 ... y_n$ are the maximum ratios of flow to saturation flow for the phases 1, 2 ...n i.e. $y = q/s$ for a given phase.

The minimum cycle length recommended is preferably 120 seconds being the maximum acceptable delay for drivers of vehicles and pedestrians.

7.6.4. **Green times :** The effective green time (g) is the sum of green period and the amber period (G) less the lost time (L) for a particular phase i.e. g = (G - L) seconds. The ratio of effective green times should be equal to the ratio of by values.

$$\frac{g_1}{g_2} = \frac{y_1}{y_2}$$

where g_1 and g_2 are the effective green times of phase 1 and 2 respectively. y_1 and y_2 are maximum of y values of phases 1 and 2 respectively. This rule can be extended to 3 or more phase operations. The effective green time for a two phase intersection can be determined from the expression :

$$g_1 = \frac{y_1}{Y}(Co - L) \text{ and } \qquad g_2 = \frac{y_2}{Y}(Co - L)$$

7.7. Control Measures

7.7.1. **Channelisation :** Channelisation involves the use of islands at intersections to guide and protect the traffic. It provides reference points within the intersection which enable drivers to better predict the path and speed of other drivers. They increase the driver's ability to avoid accidents and congestion. Channelised Islands should be at least 4.7m^2 and preferably 7.1m^2 in area, not less than 2.4 m and preferably 3.6 m on any side after rounding of corners if triangular and at least 1.2 m and preferably 3.6 m if elongated.

7.7.2. **Carriageway Markings :** Carriageway markings and direction signs should be located sufficiently in advance of the intersection to enable drivers to select and follow the lane and path they should take through the intersection, Markings at intersections can be a combination of centre lines, turn markings, lane markings, stop lines, route direction arrows, pedestrian crossings etc. (Fig. 7.7)

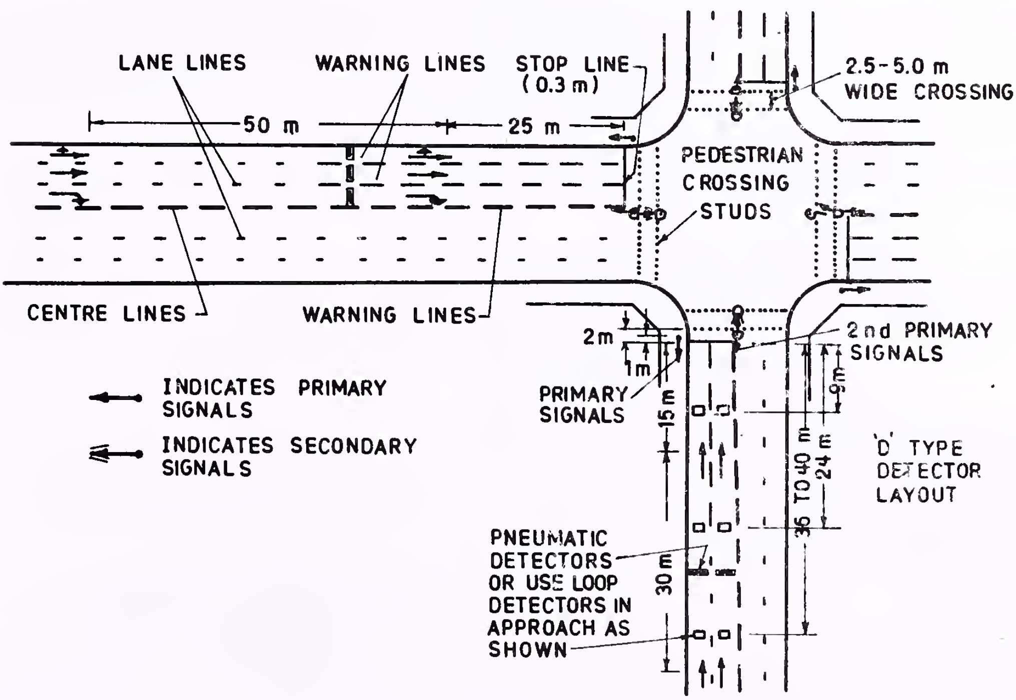


Fig. 7.7. Signalised Intersection Layout and Carriageway Markings

7.8. Other Aspects

7.8.1. **Visibility :** The safety of traffic can be ensured only if the visibility is full and unimpeded along both roads. Any obstruction should be clear of the minimum visibility triangle for a height of 1.2 m above the roadway.

The intersection should be planned and located to provide as much sight distance as possible. In achieving a safe highway design, there should be sufficient sight distance for the driver on the minor highway to cross the major highway without requiring the approaching traffic to reduce speed.

7.8.2. **Installation of signals :** The installation of signals and other technical aspects of road traffic signals are discussed in detail in section II of IRC : 93-1985 and may be referred. A typical layout is shown in Fig. 7.8.

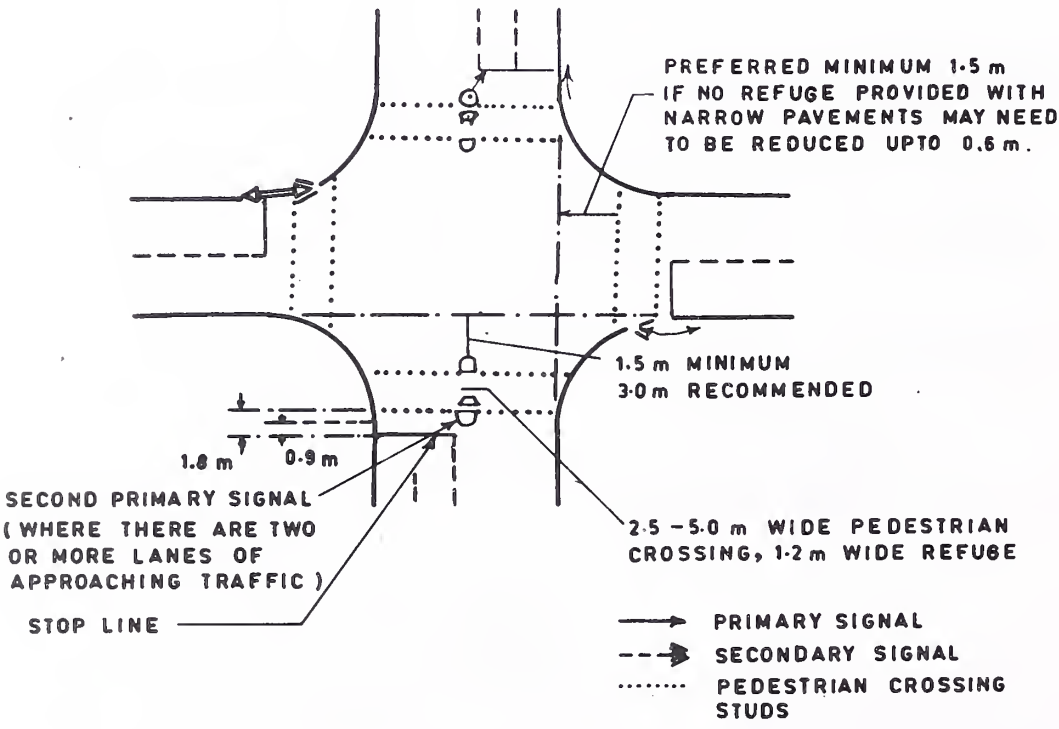


Fig. 7.8. Typical Layout of Traffic Signal Installations

8. SPECIAL CONSIDERATIONS IN URBAN AREAS

8.1. Control of Access

This aspect is discussed in detail in IRC : 62-1976 "Guidelines for Control of Access on Highways". To ensure safe and efficient circulation of traffic, which serves the various land uses adequately and ensures logical community development, the network of roads in an urban area has to be divided into different sub-systems each serving a particular function or a purpose. The principal factors to be considered in designating roads into different categories are the travel desire lines, access needs of adjacent properties, network pattern and land use. For the purpose of these guidelines, urban highways/streets are considered to be divided into the following types :

(i) Arterial highways/streets,

(ii) Sub-arterial streets,

(iii) Collector streets; and

(iv) Local streets.

8.2. Spacing of Intersections

Although standards for the location of access points depend largely on the needs of an area, the following guidelines which are indicative of good practice, may be followed as far as possible.

8.2.1. Spacing between intersections should have regard to the relevant geometric design and traffic requirements, such as the type of traffic, length of right-turn or speed change lanes etc.

As a rough guide, the suggested minimum spacing along various types of roads is given below :

(i)	Arterial highways/streets	500 metre
(ii)	Sub-arterial streets	300 metre
(iii)	Collector streets	150 metre
(iv)	Local streets	Free access

Where necessary, a greater distance than given above should be adopted, as for example between junctions with linked traffic signals.

8.2.2. On arterial streets signals should preferably be linked to have a progressive system, premitting continuous movement of vehicles at a planned speed of travel. As far as possible, all such intersections should have approximately the same spacing.

8.2.3. Apart from regular intersections, limited number of access points with intervening streets may be permitted at a spacing closer than mentioned in para 8.2.1 provided only left turns to and from the main street are permitted.

8.2.4. The location and spacing of all major points of access including accesses to bus terminals, railway stations, parking areas etc. should be carefully planned so as to ensure safety and freedom from congestion. Bus bays should not be located too close to intersections. It is desirable that they are located 75m from the intersection on either side, preferably on farther side of the intersection.

8.2.5. On arterials, direct access to residential plots is not to be permitted. Drive-ways may, however, be permitted on a restricted basis for commercial and industrial complexes and other public locations when these are major generators of traffic. Right turn from these driveways should not be permitted unless the crossing fulfills the spacing criteria given in para 8.2.1. Moreover, adequate road geometrics should be provided to enable safe operation of vehicles.

8.2.6. On sub-arterials, direct access to residential property should be granted only where alternative access cannot be provided at a reasonable cost. Direct access to commercial and industrial properties may be allowed.

8.2.7. On collector streets, access to abutting properties may be allowed to a limited extent keeping in view the safety of traffic.

8.2.8. On local streets, which will have no through traffic, access to abutting properties can be freely given.

8.3. Pedestrians at Junction

Pedestrian Crossings (e.g. at grade crossings, subways) should be designed at intersection in accordance with the "Guidelines on Pedestrian Facilities" (IRC : 103-1988).

8.4. Cycle Tracks

8.4.1. It is desirable to segregate cycle traffic at intersections also since intersections are dangerous, accident-prone locations. This could be achieved by a suitable system of multi-phase signalisation.

8.4.2. Where cycle tracks are not segregated, suitable safety measures at intersections should be adopted, such as provision of separate lanes for cycle traffic, provision of cycle boxes ahead of stop line, and provision of turning cycle paths in conjunction with signalisation. These methods are shown in Fig. 8.1. Cyclist crossing must always be marked in accordance with IRC : 35-1970.

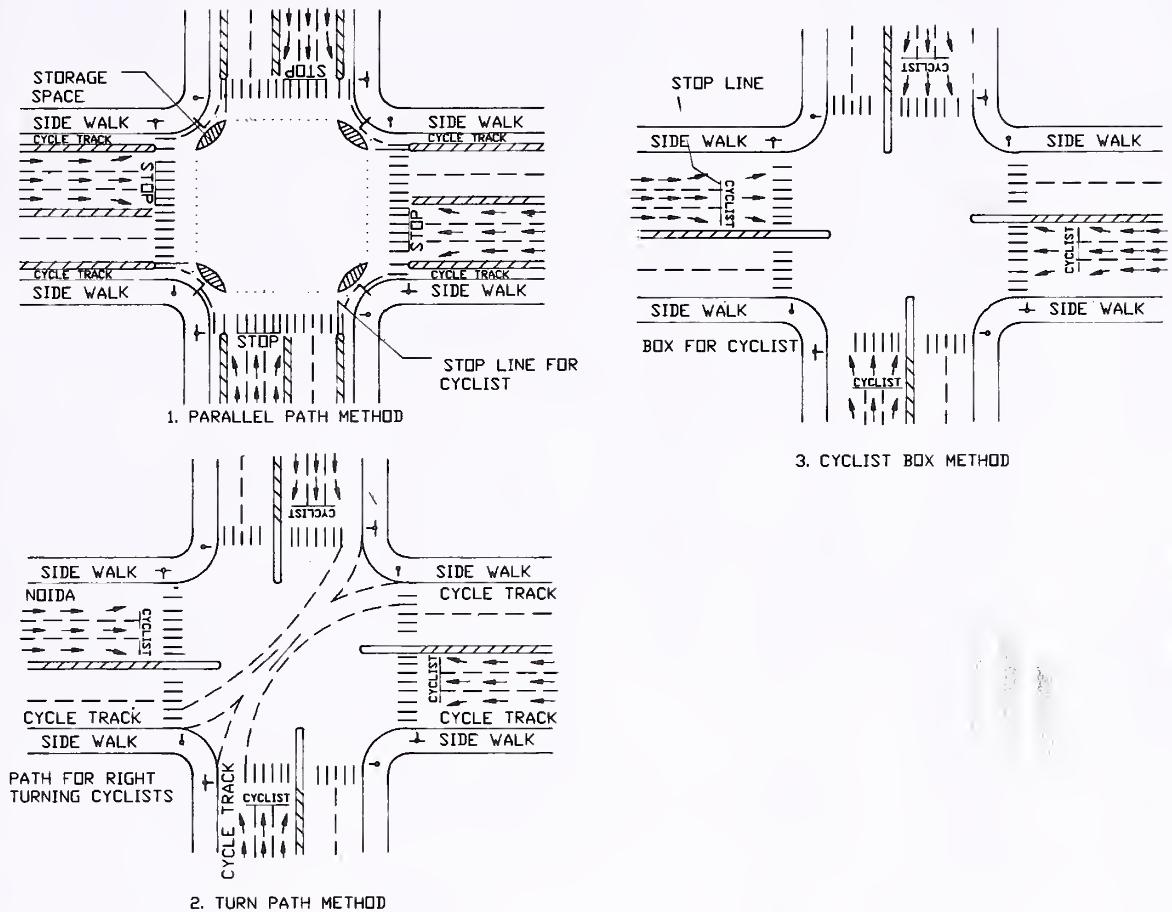


Fig. 8.1. Segregation of Cycle Traffic at Road Intersection

8.4.3. At signalised intersections, a separate lane for cycle traffic alongwith a reservoir space between pedestrians crossing zone and stop line for motorized vehicles provides an efficient regulation. Fig. 8.2 shows the details of such layout. A separate cycle track of minimum 1.2 m width should be provided in the approaches of intersection. There would be two stop lines, one for the motor vehicles and other for cyclists such that the cyclists can wait in the reservoir space, for signal to turn green. The signs to be provided and their locations as indicated in Fig. 8.2 are self explanatory.

8.5. Lighting & Sign Posting

The adequate lighting of intersections especially with channelising island is essential in urban areas. Lighting should be so done as to make the entire island and intersecting area visible even in bad weather. The aspect of lighting is discussed in more detail in Chapter 9.

The safety of intersection also depends on adequate carriageway markings, traffic signs and delineators. These should be provided in accordance with IRC : 35-1970, IRC : 67-1977 and IRC : 79-1981.

9. LIGHTING, DRAINAGE, UTILITIES AND LANDSCAPING OF INTERSECTION

9.1. Lighting at Intersections

Lighting affects the safety of highways and street intersections and the ease and comfort of traffic operations. Statistics indicate that the evening accident rate is higher than that during day-light hours. This fact, to a large degree, may be attributed to impaired visibility. In urban and suburban areas where there are concentrations of pedestrians and roadside and intersectional interference, fixed source lighting tends to reduce accidents.

9.1.1. Whether or not rural intersections at grade should be lighted depends on the layout and the traffic volumes involved. Intersections that normally do not require channelization are frequently left unlighted. On the other hand, intersections with substantial channelisation, particularly multi-road layouts and those designed on a broad scale, often are lighted. It is especially desirable to illuminate large-scale channelized intersections. There is need to obtain a reduction in the speed of vehicles approaching some intersections. The indication of this need should be definite and visible at a distance from the intersection that is beyond the range of headlights. Illumination of the intersection with fixed-source lighting accomplishes this. The planned location of intersection luminairs supports poles should be designed to present the least possible hazard to out-of-control vehicles. The breakaway support base should not be used within the limits of an at-grade intersection, particularly in densely developed areas with adjacent sidewalks.

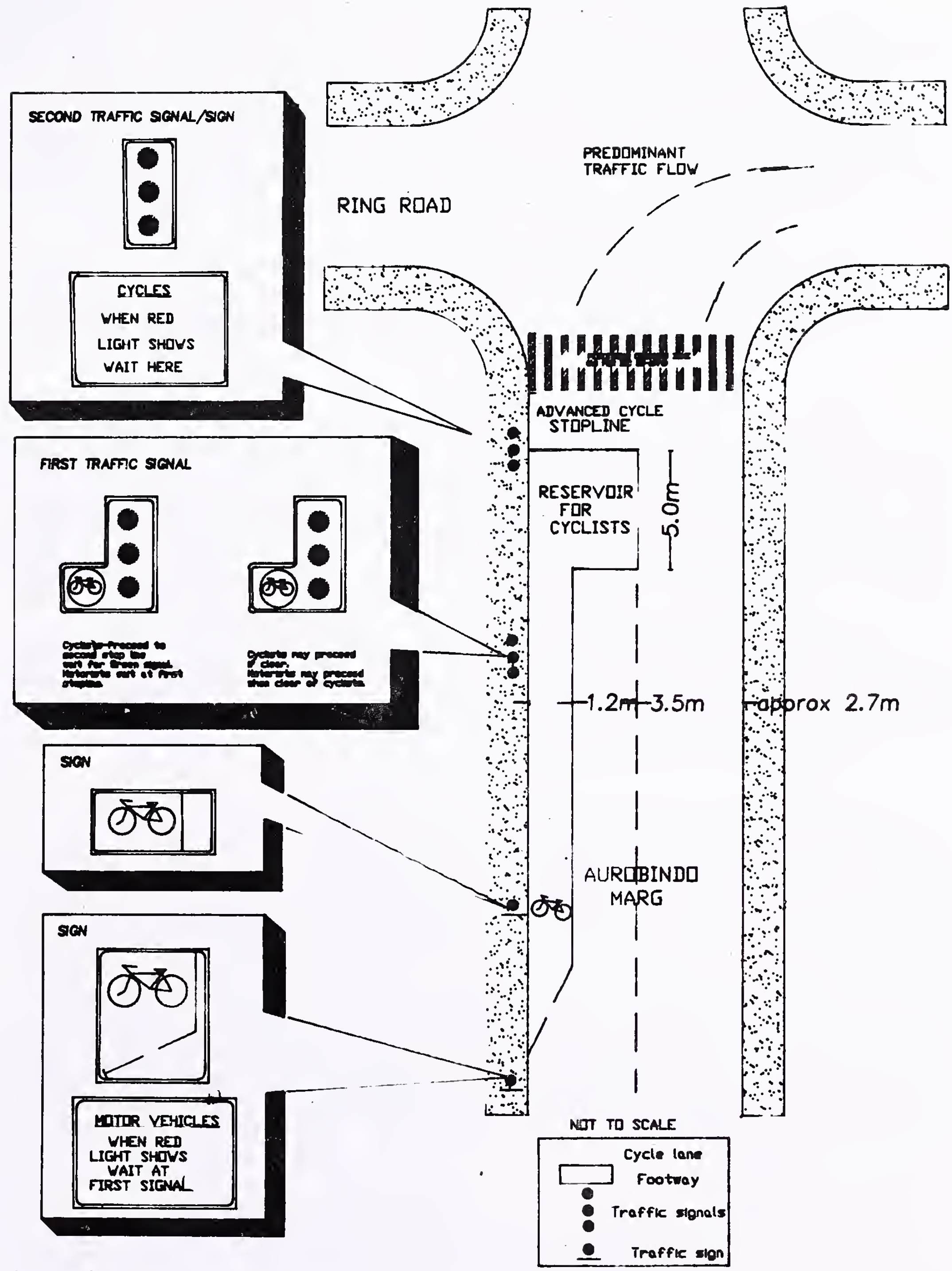


Fig. 8.2. Cyclists Crossing at Signalised Intersection

9.1.2. To minimize the affect of glare and to provide the most economic lighting installation luminaries are mounted at heights of at least 9 m. Lighting uniformity is improved with higher mounting heights, and in most cases mounting heights of 10 m to 15 m are usually preferable. High mast lighting, special luminaries on masts of 80m or more in height, is used to light large highway areas such as inter-changes. This lighting furnishes a uniform light distribution over the whole area and may provide alignment guidance. However, it also has a disadvantage in that the visual impact from spurious light is increased to the surrounding community.

Lighting standards (poles) should be placed outside the clear zones whenever practical. Where poles are located within the clear zone regardless of distances from the travelled way, they should be designed to have the suitable impact attenuation features; normally a breakaway design is used.

On a divided highway the street lighting standards may be either in the median or on the left. Where lighting standards are located on the left, the light source is closer to the more heavily used traffic lanes. However, with median installation, the cost is generally lower and illumination is greater on the high-speed lanes. On median installations, dual-mast arms should be used, for which 12m or 15m mounting heights are favoured. These should be protected with suitable longitudinal barrier. On narrrow medians, it is usually preferable to place the lighting standards so that they are integral with the median barrier.

Where highway lighting is being considered for future installation considerable savings can be effected through design and installation of necessary conduits under pavements and kerbs as part of initial construction.

9.2. Utilities

All highways and street intersections, whether upgraded within the existing right-of-way or entirely on new right-of-way, generally entail adjustment of utility facilities. Although utilities generally have little effect on the geometric design, full consideration should be given to measures, reflecting sound engineering principles and economic factors, necessary to preserve and protect the integrity and visual quality of the highway or street, its maintenance efficiency and the safety of traffic. Depending on the location of a project, involved utilities could include (1) sanitary sewers; (2) water supply lines; (3) oil, gas, and petroleum products pipelines; (4) overhead and underground power and communication lines; and (5) drainage and irrigation lines. IRC : 98-1988 "Guidelines on Accommodation of Underground Utility Services Along and Across Roads in Urban Areas" provides guidelines in this regard.

9.3. Drainage

9.3.1. Highway drainage facilities provide for carrying water across the right-of-way and for removal of storm water from the road itself. These facilities include bridges, culverts, channels, kerbs, gutters and various types of drains. Hydraulic capacities and locations of such structures should be designed to secure as low a degree of risk of traffic interruption by flooding as is consistent with the importance of the road, the design traffic service requirements, and available funds.

9.3.2. Drainage inlets should be so designed and located as to limit the spread of water on traffic lanes to tolerable widths. Because grates may become blocked by trash accumulation kerb openings or combination inlets with both grate and kerb openings are advantageous for urban conditions. Grate inlets and depressions and kerb-opening inlets should be located outside the through traffic lanes to minimise the shifting of vehicles attempting to avoid riding over them. Inlet grates should also be designed to accommodate bicycles and pedestrian traffic where appropriate. Inlets should be located so that concentrated flow and heavy sheet flow will not cross traffic lanes. Where pavement surfaces are warped, as at cross streets, surface water should be intercepted just before the change in cross slope. Also, inlets should be located just upgrade of pedestrian crossing.

9.4. Landscaping

Landscape development at intersection should be in keeping with the character of the highway and its environment. However, requirements of clear sight distance must be kept in mind. Programmes include the following general areas of improvement : (1) Preservation of existing vegetation (2) Transplanting of existing vegetation where feasible (3) Planting of new vegetation, (4) Selective clearing and thinning, and (5) Regeneration of natural plant species and material.

The objectives in planting or the retention and preservation of natural growth on roadsides are closely related. In essence, they are to provide (1) Vegetation that will be an-aid to aesthetics and safety, (2) Vegetation that will aid in lowering construction and maintenance costs, and (3) Vegetation that creates interest, usefulness, and beauty for the pleasure and satisfaction of the travelling public.

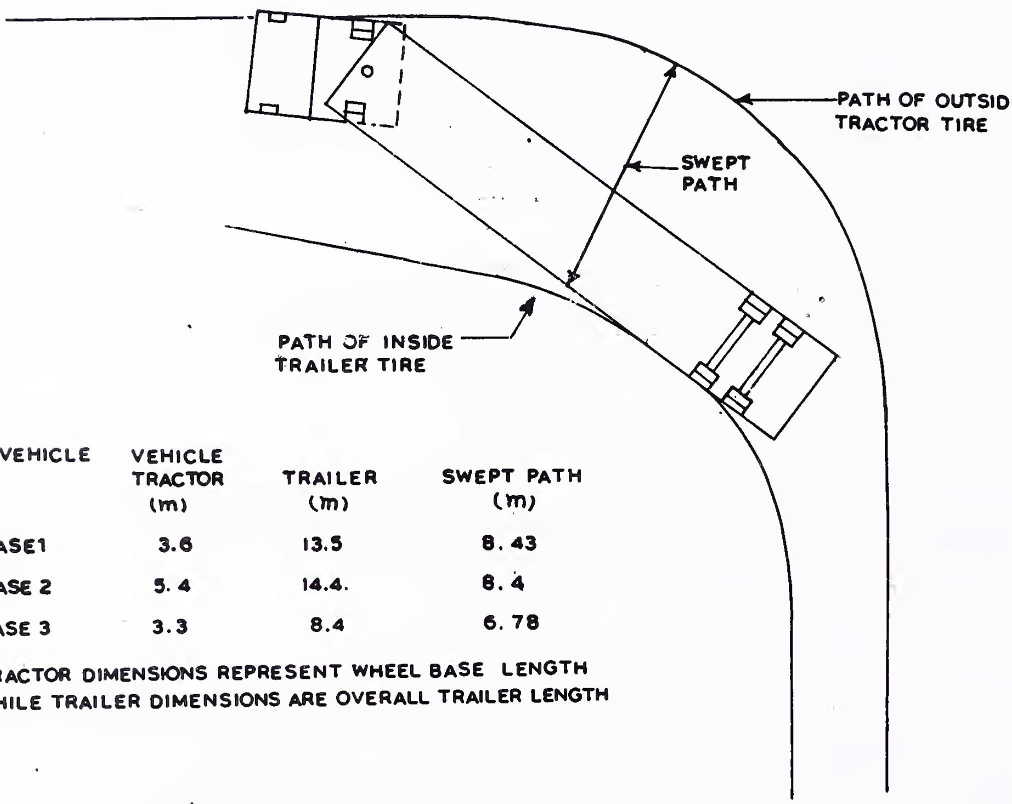
Landscaping of urban intersections assumes additional importance in mitigating the many nuisances associated with urban traffic. Landscaping can reduce this contribution to urban plight. For more detailed guidelines, reference to the IRC Manual on "Landscaping of Roads", Published in 1979 may be made.

9.5. Right of Way

The right of way for the road and area required for an intersection depend largely on the type of highway topography and overall standards of intersection development. The required sight line at the intersection and impact on property access occurring with the provision of intersection would also have to be considered. While deciding upon the area required to be acquired for the intersection, drainage aspect and future planning, in particular the need for the upgradation of intersection to a grade separated facility as dictated by traffic projections, shall also be considered. The area shall be kept free from encroachments to facilitate the implementation of future planning.

LAYOUT OF CURVES AT THE INTERSECTION

For proper design of curves for intersections a full appreciation of the turning path of vehicle is essential. Fig. I-1 shows the swept path of semi-trailer negotiating a 90° turn at low speed. Figs. I-2, I-3, I-4, I-5 and I-6 show details of wheel path and body overhang path of large car, single unit truck, semi-trailer, and large truck-trailer combination respectively. Tamplates of these paths made to appropriate scale should be used to check layout of curves at intersections. Figs. I-7 to I-10 show typical layouts of curves for a 90° turn using different vehicles and curves.



VEHICLE	VEHICLE TRACTOR (m)	TRAILER (m)	SWEPT PATH (m)
CASE1	3.6	13.5	8. 43
CASE 2	5. 4	14.4.	8. 4
CASE 3	3.3	8.4	6. 78

TRACTOR DIMENSIONS REPRESENT WHEEL BASE LENGTH WHILE TRAILER DIMENSIONS ARE OVERALL TRAILER LENGTH

Fig. I-1. Swept Path Width for Various Truck Vehicles — Low speed Offtracking in a 90° Intersection Turn

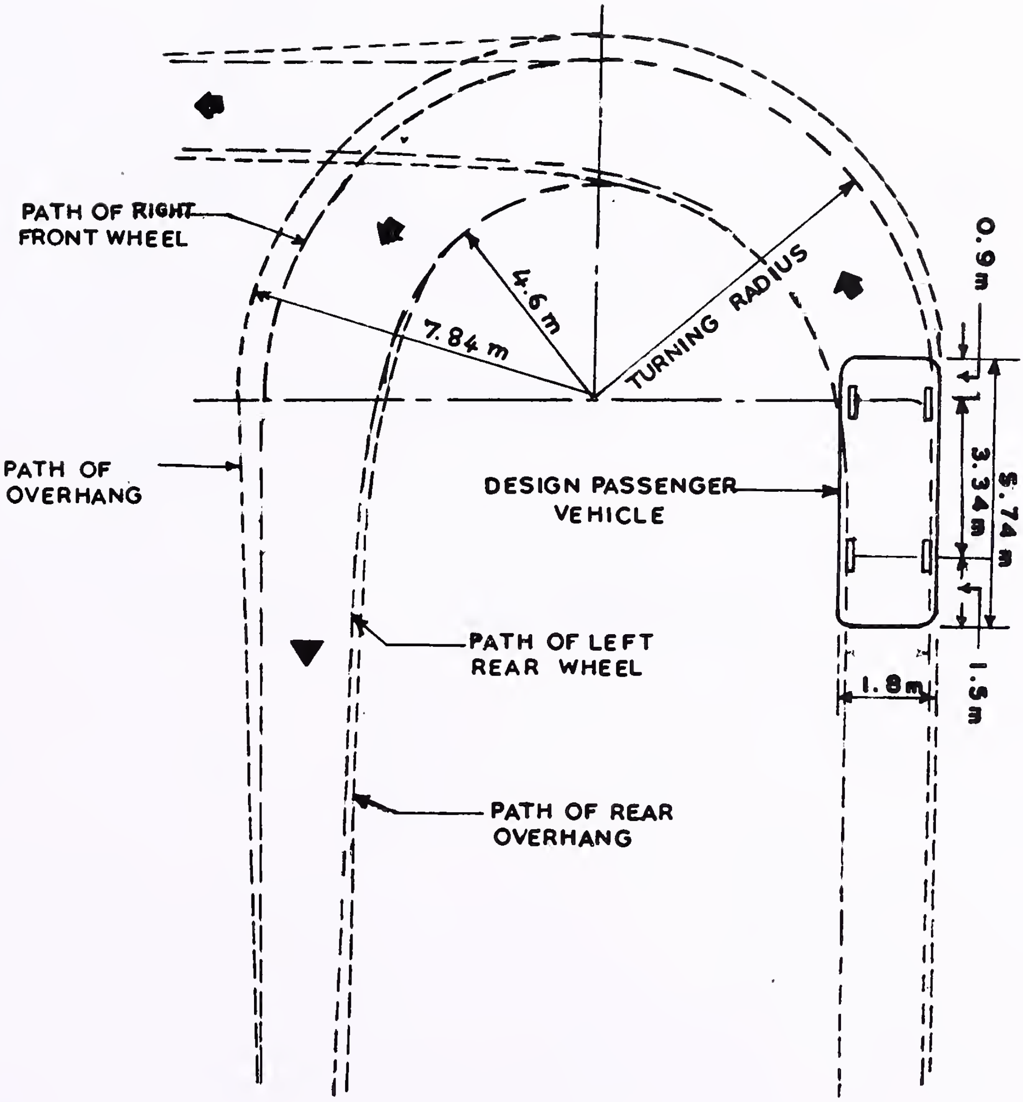


Fig. I-2. Minimum Turning Path for Passenger Car Design Vehicle (P)

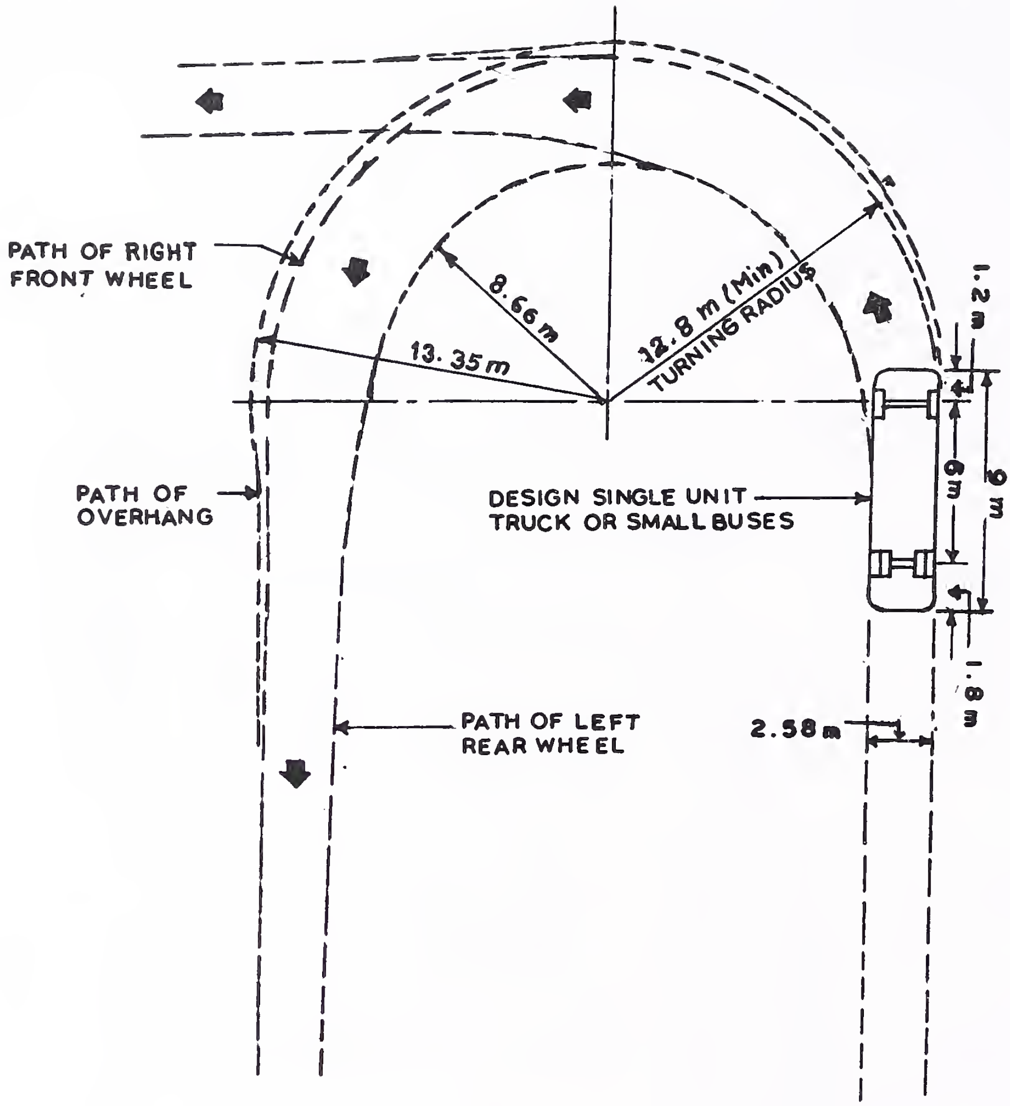


Fig. I-3. Minimum Turning Path for Single unit Truck Design Vehicle (su)

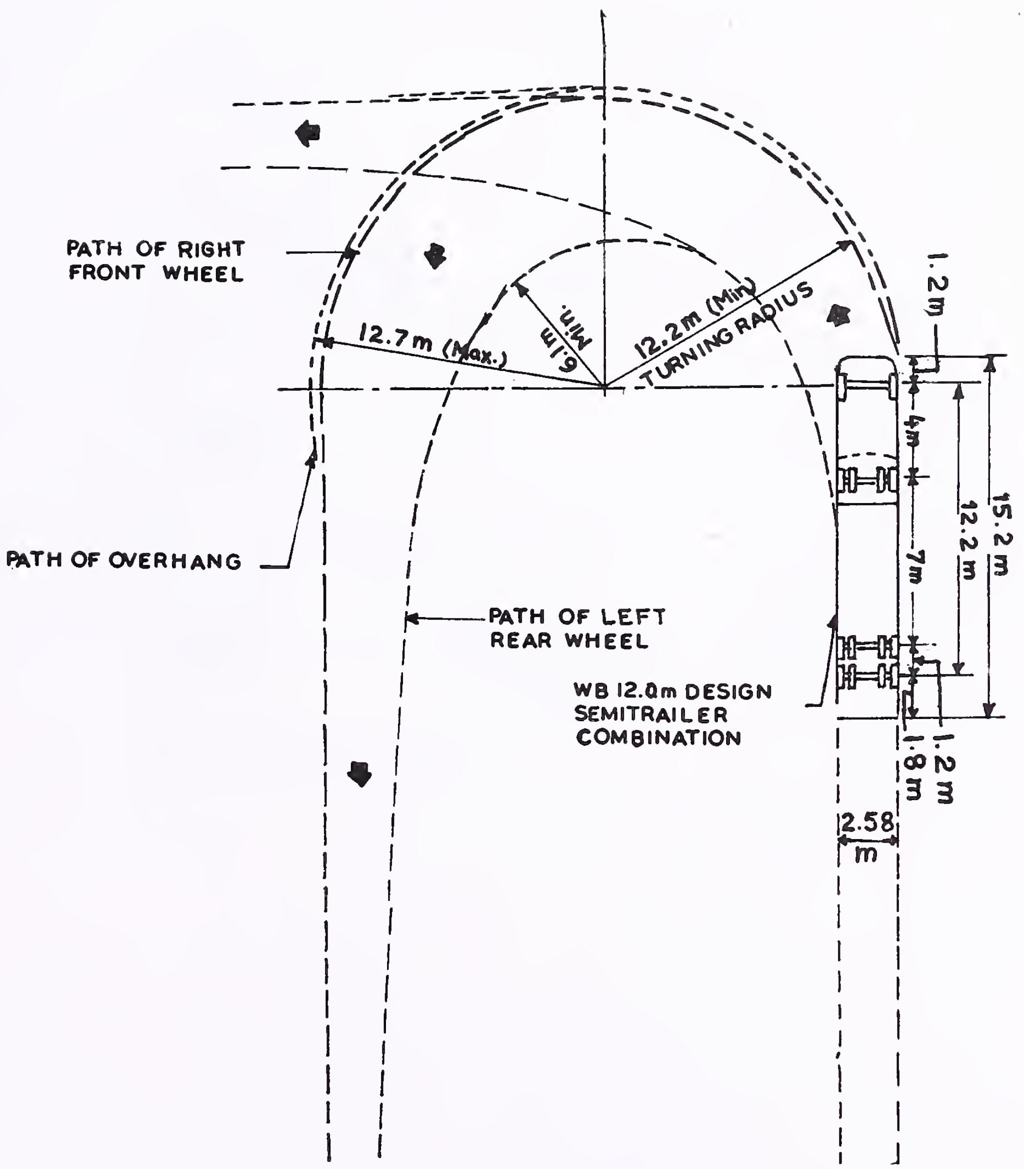


Fig. I-4. Minimum Turning Path for Semi-Trailer (WB-12.0) Design Vehicle

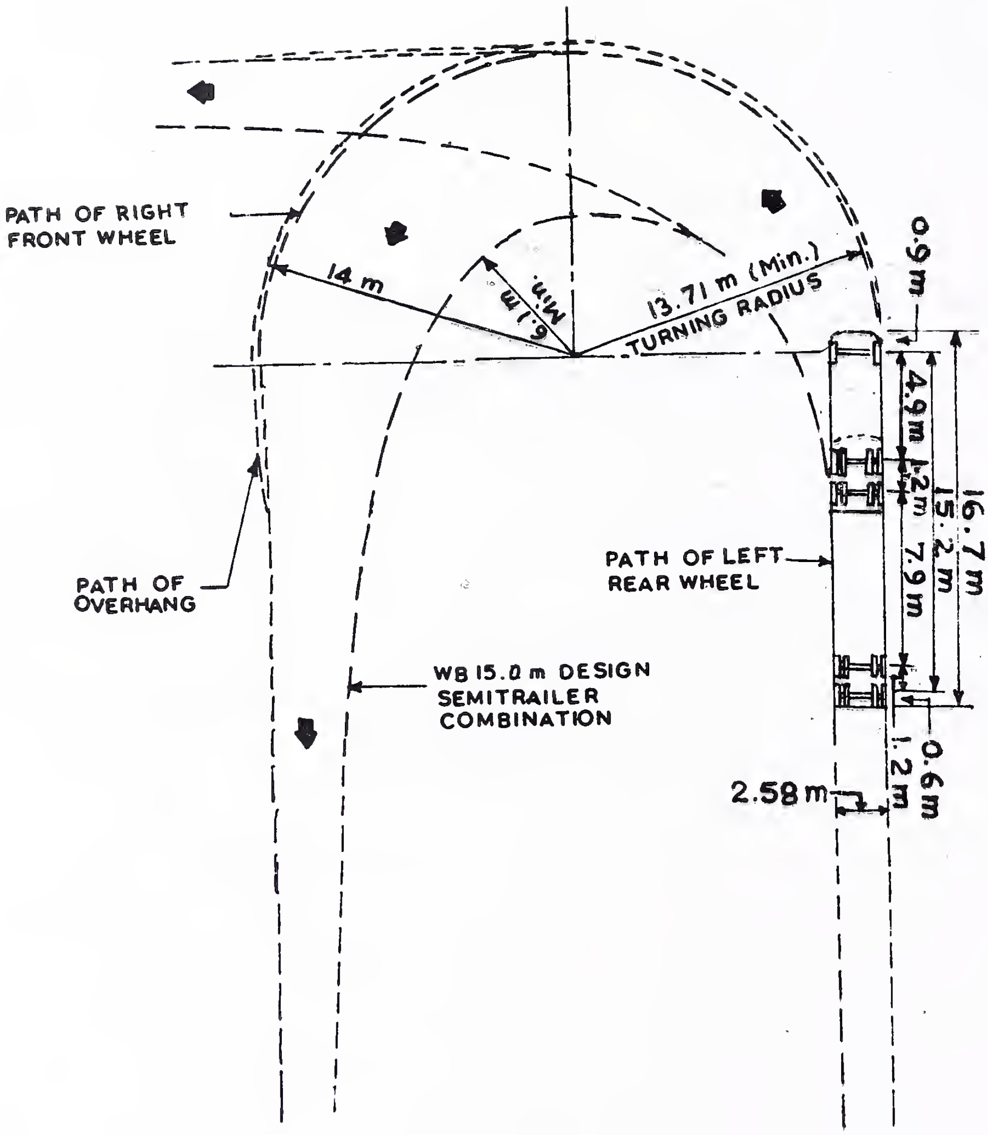


Fig. I-5. Minimum Turning Path for Semi Trailer (WB-15.0) Design Vehicle

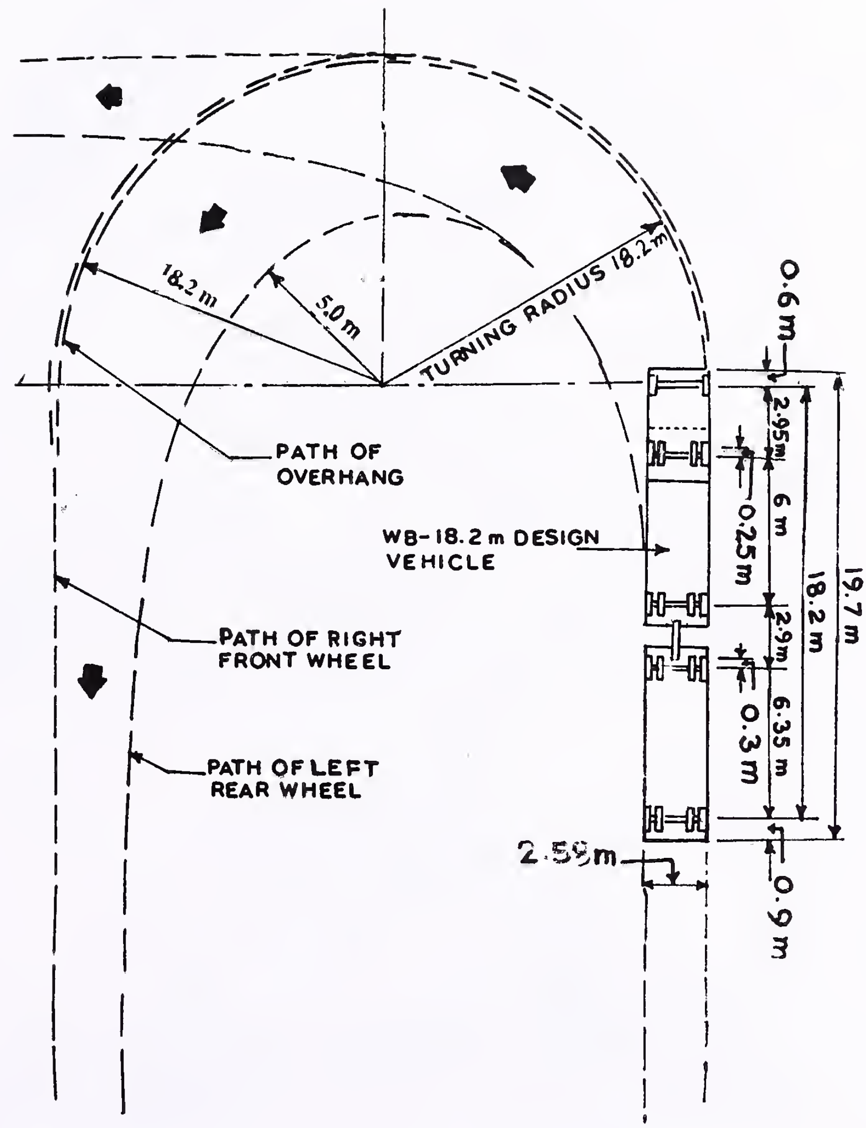


Fig. I-6. Minimum Turning Path for Truck Trailer (WB-18.0) Design Vehicle

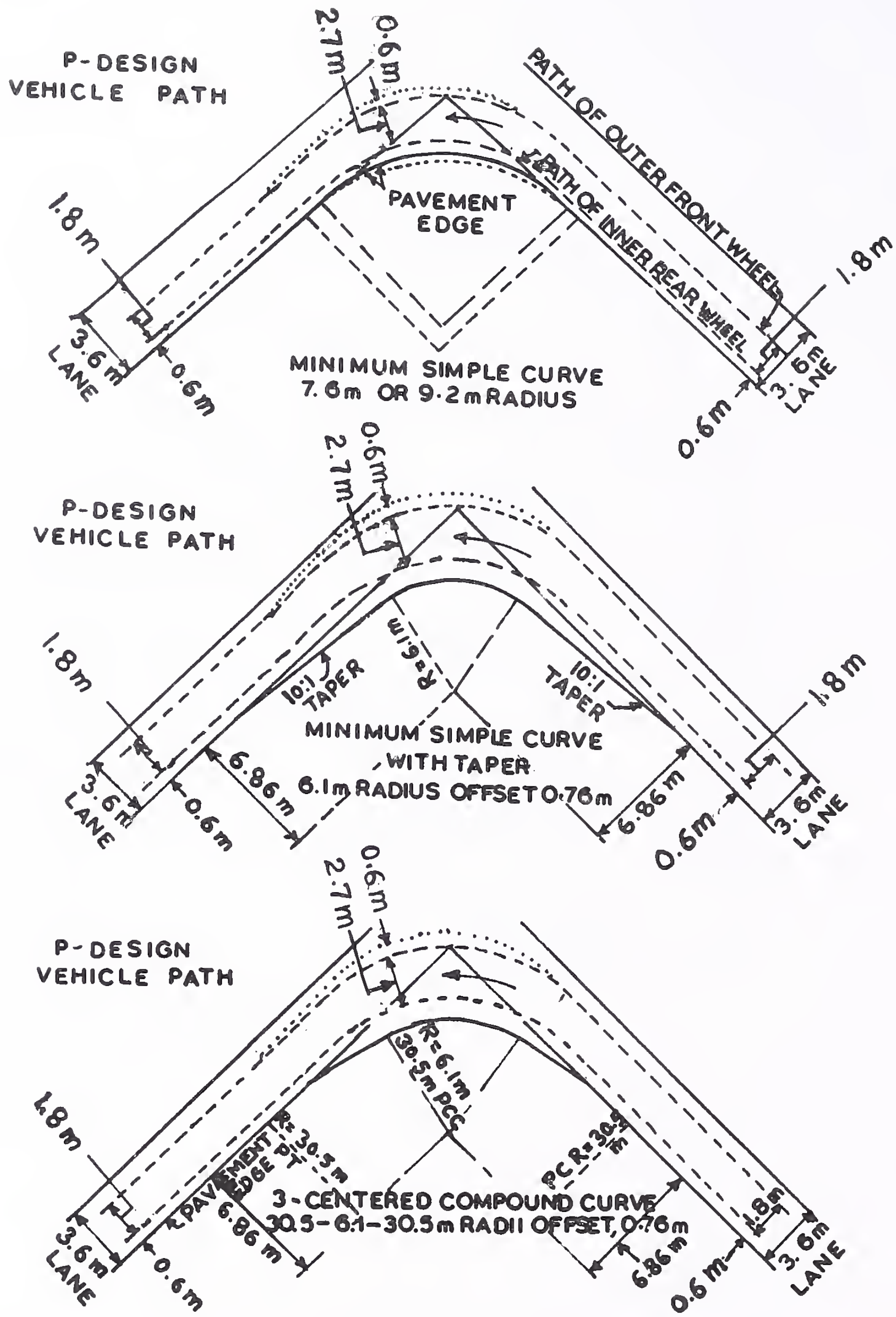


Fig. I-7. Minimum Designs for Passenger Vehicles for 90° turn

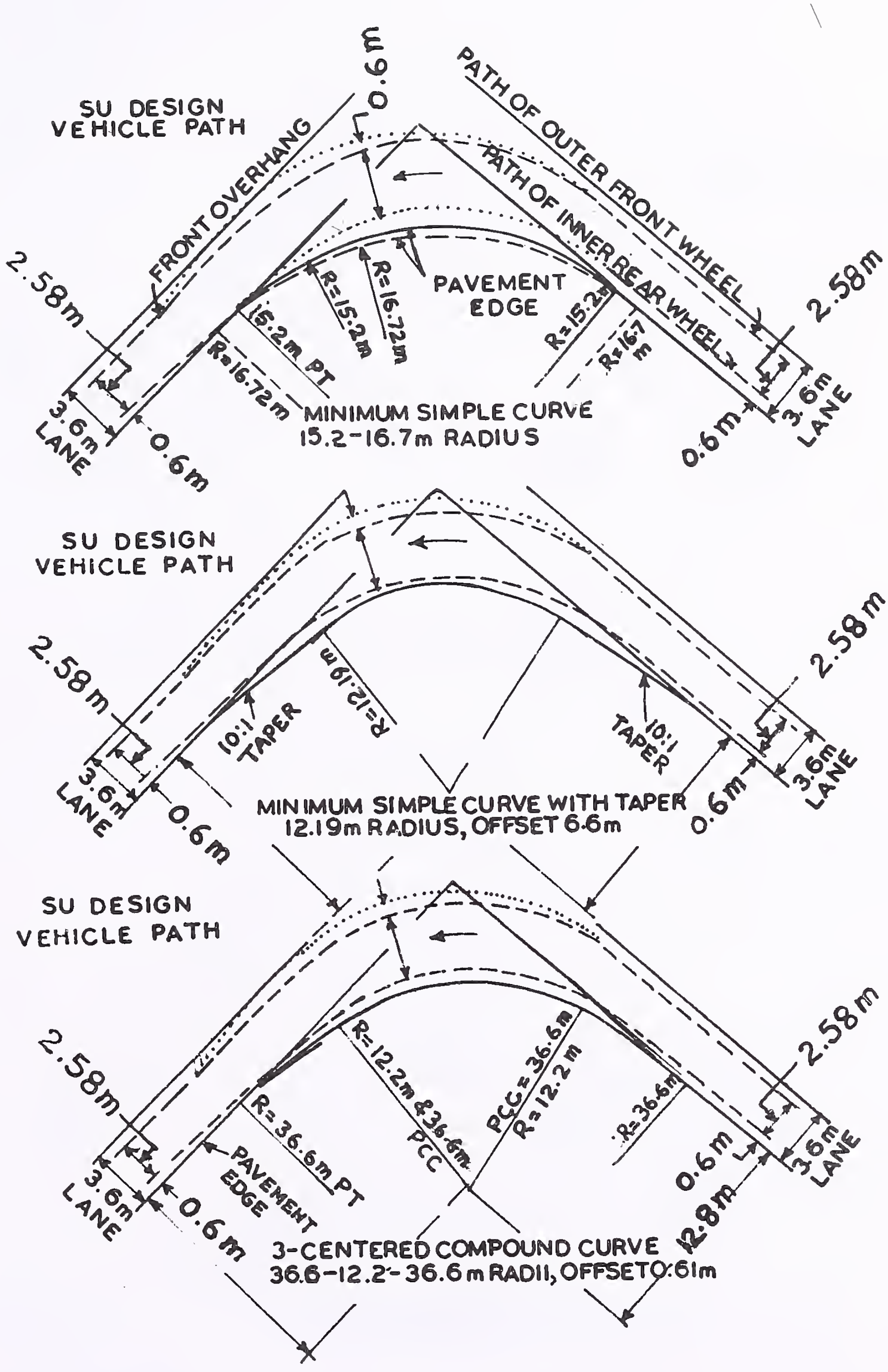
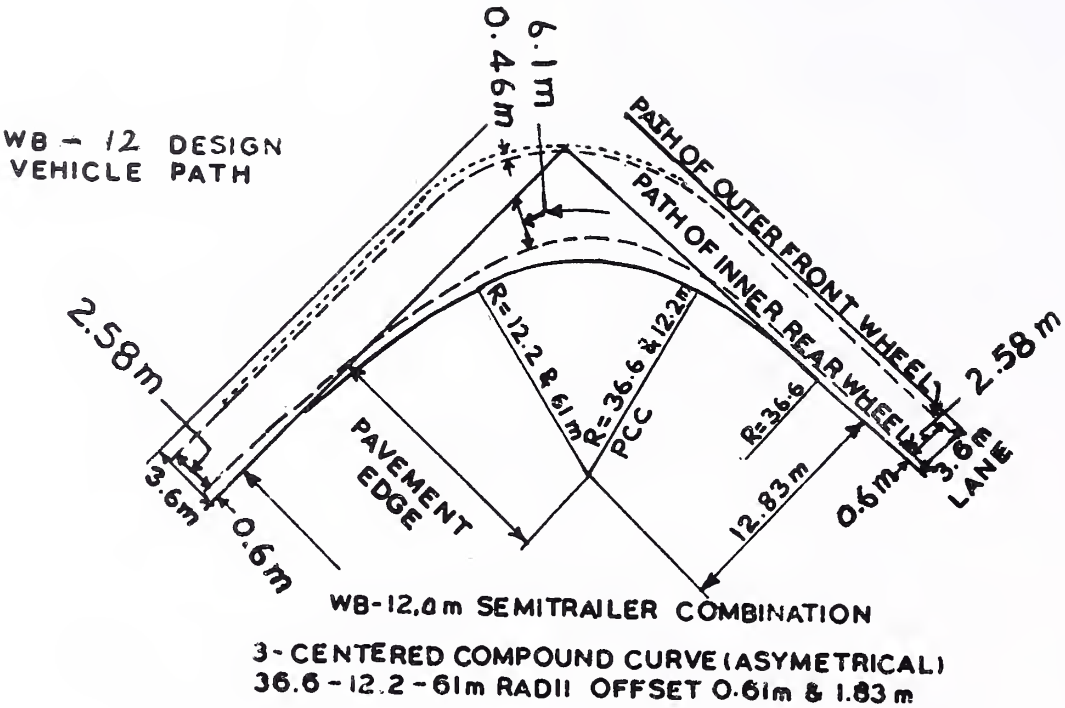


Fig. I-8. Minimum Designs for Single Unit Trucks and Buses for 90° Turn



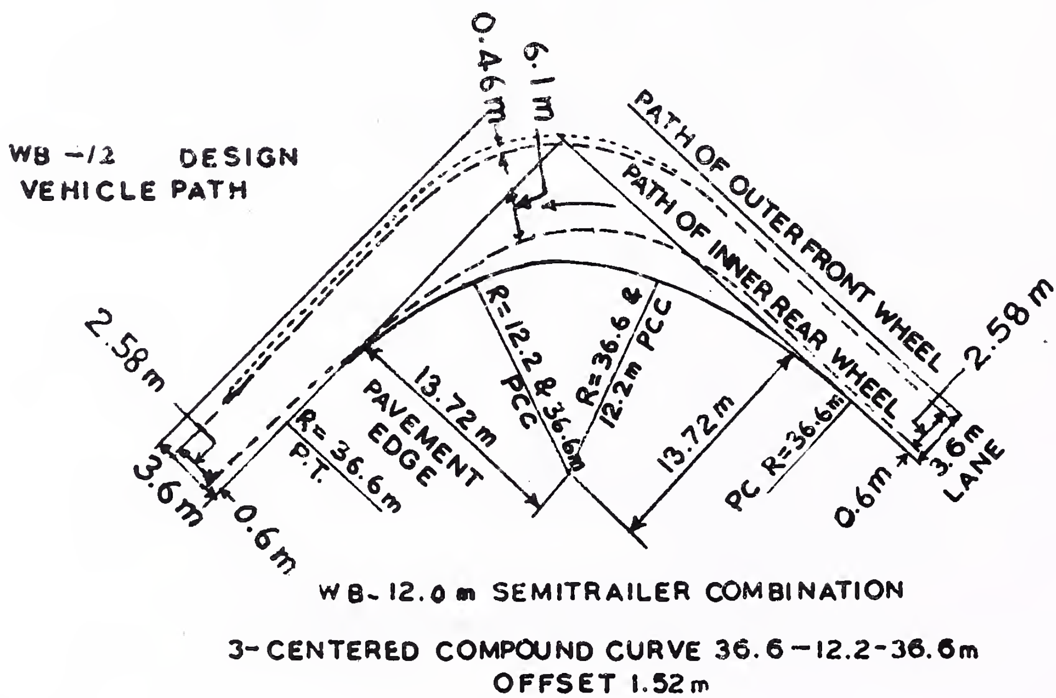


Fig. I-9. Minimum Designs for Semi-trailer Combinations (WB-12.0) Design Vehicle Path for 90° Turn

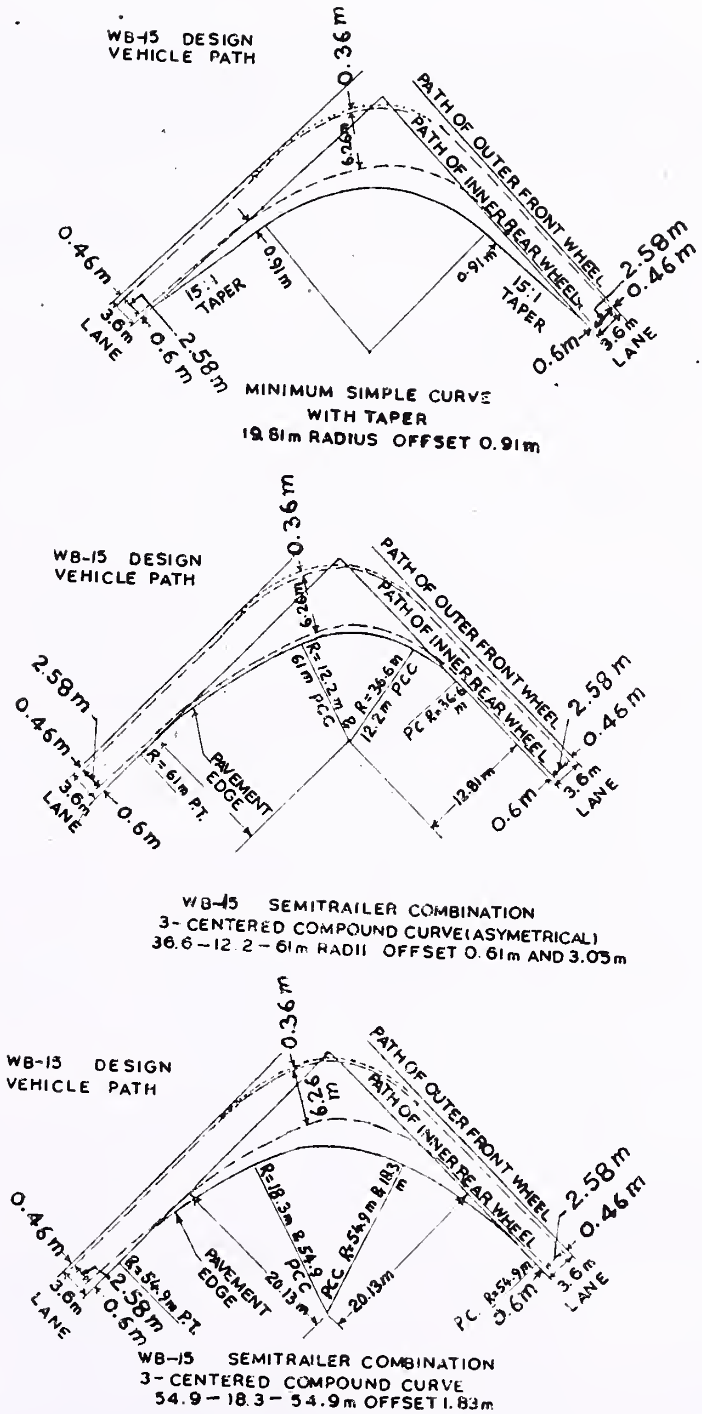


Fig. I-10. Minimum Designs for Semi-trailer Combinations (WB-15)
Design Vehicle Path for 90° Turn

The effect of angle of intersection on turning path and width occupied by vehicles with turning radii is indicated in Table I-1. This is useful in determining the desirable width of lane and corresponding radii of curve. In Table I-1 d_1 and d_2 are the widths occupied by the turning vehicles on the main street and on cross-street respectively while negotiating turns through various angles. Both dimensions are measured from the left hand kerb to the point of maximum turn.

The minimum radii of curves for various angles of intersection based on dimension in Table 4.5 are given in Table I-2.

2. Layout of three-Centered Compound Curve

Layout of 3-centered curves require calculations of offsets and co-ordinates and centre of curves. This can be mathematically derived as below :

Let	R_1 = First Radius	x_1 = First Offset
	R_2 = Middle Radius	x_2 = Second Offset
	R_3 = Third Radius	α = Angle of turn

Fig. I-11(a) shows the curve and its centres and Fig. I-11(b) is an enlarged plan of the required portion to illustrate clearly.

Determination of the position of 'O' the centre of radius R_2 the middle radius ; P_1 Q is an arc drawn with radius R_1, Q R with radius R_2 and R S with radius R_3 having offset x_1 at point Q and offset x_2 at point R.

$$P_1O_1 \parallel P_2O_2 \text{ or } O_2 d$$

$$\angle P_1O_1Q = \angle PO_2Q = Q_1 \qquad \text{... Fig. I-11(a)}$$

$$QE = C_1P_1 = x_1 = \text{First Offset} \qquad \text{... Fig. I-11(b)}$$

$$C_1P_1 = R_1 - R_1 \cos\theta_1$$

$$\text{or } x_1 = R_1 - R_1 \cos\theta_1$$

$$\text{or } \cos\theta_1 = \left(1 - \frac{x_1}{R_1}\right) = 1 - \frac{\text{Offset}}{\text{Outer Radius}}$$

$$O_2D = O_2C_2 + C_2D$$

$$= R_2 \cos {}_{-1} + x_1$$

$$O_2D = R_2\left(1 - \frac{x_1}{R_1}\right) + x_1$$

Therefore distance between A'O_2 line AO.

$$y_1 = R_2\left(1 - \frac{x_1}{R_1}\right) + x_1$$

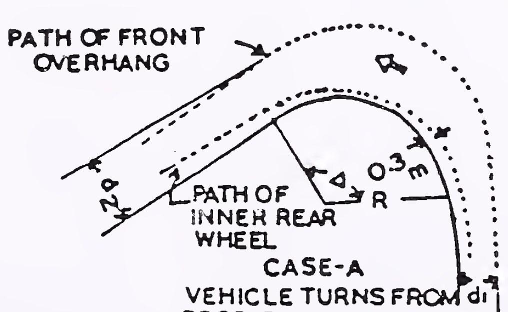
$$= R_2 + x_1 - x_1 \frac{(R_2)}{R_1}$$

or middle radius + offset fraction of offset which is equal to offset multiplied by the ratio of middle radius to outer radius

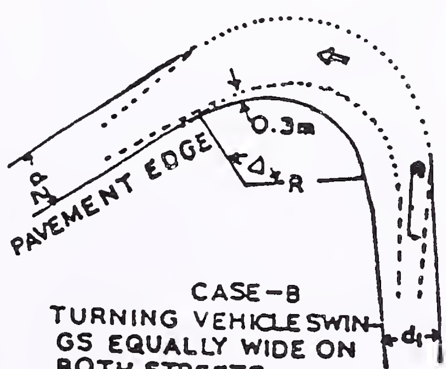
Table I-1. Urban Areas Cross-street width occupied by turning vehicle for various angles of intersection and kerb radii

	Design Vehicle	d_2(m) for cases A and B where									
		R = 4.5m		R = 6m		R = 7.5 m		R = 9 m		R = 12 m	
		A	B	A	B	A	B	A	B	A	B
30°	SU	4.26	3.95	4.26	3.95	3.95	3.95	3.95	3.95	3.95	3.95
	BUS	6.69	5.17	5.78	5.17	5.78	5.17	5.78	5.17	5.47	5.17
	WB 40	4.26	4.26	4.26	4.26	4.26	4.26	4.26	4.26	4.26	4.26
	WB 50	6.00	5.17	6.00	5.17	6.00	5.17	5.78	4.87	5.47	4.87
60°	SU	5.78	4.87	5.78	4.78	5.17	4.5	4.87	4.5	4.26	4.26
	BUS	8.55	6.38	7.9	6.0	7.23	6.0	6.99	5.78	6.69	5.47
	WB 40	7.23	5.78	6.69	5.78	6.38	5.78	5.78	5.47	5.17	4.87
	WB 50	9.42	6.69	8.2	6.38	8.55	8.82	7.6	5.78	6.63	5.47
90°	SU	7.90	6.00	6.99	5.47	5.78	4.87	5.17	4.5	3.95	3.95
	BUS	11.55	6.99	10.0	6.69	9.12	6.63	7.6	6.38	6.38	5.47
	WB 40	9.43	6.69	8.2	6.38	6.99	6.38	5.78	5.47	5.17	4.87
	WB 50	12.80	6.69	11.27	7.29	10.34	6.69	8.83	6.38	6.69	5.47
120°	SU	10.34	6.69	8.2	5.78	6.38	5.47	5.17	4.78	3.95	3.95
	BUS	13.98	8.55	12.2	7.6	9.73	6.99	7.92	5.78	5.78	5.47
	WB 40	12.23	6.99	8.83	6.69	7.23	6.69	5.78	5.47	5.17	4.87
	WB 50	15.3	8.83	13.10	8.55	10.97	8.2	9.12	7.9	6.69	5.47
150°	SU	12.2	7.6	9.75	6.30	6.69	5.79	5.17	4.87	3.65	3.65
	BUS	14.59	8.55	12.2	7.6	9.75	6.99	6.69	5.47	5.17	4.87
	WB 40	11.86	7.23	8.83	6.69	6.99	6.69	5.78	5.47	51.7	4.87
	WB 50	16.11	9.43	14.0	8.55	10.97	8.2	8.55	7.92	6.69	5.47

Notes : P Design Vehicle Turns within 3.6 m width where R = 4.5 m No. parking Lanes on Either Street



CASE-A
VEHICLE TURNS FROM PROPER LANE AND SWINGS WIDE ON CROSS STREET
d_1=3.6m, d_2 is variable



CASE-B
TURNING VEHICLE SWINGS EQUALLY WIDE ON BOTH STREETS
d_1=d_2 Both variable

Table I.2. Minimum Radii of Edge of Pavement for Turns at Intersections

Design Vehicle	Angle of Turn Degrees	Simple Curve, Radius Metres	Curve with Taper		
			Radius metre	Offset metre	Taper
P	30	18.3	—		
SU		30.5	—		
WB-12.0		45.7	—		
WB-15.0		61.0	—		
P	45	15.2	—		
SU		22.9	—		
WB-12.0		36.6	—		
WB-15.0		51.8	36.6	0.61	15 : 1
P	60	12.2	—	—	—
SU		19.3	—	—	—
WB-12.0		27.4	—	—	—
WB-15.0		—	28.96	0.61	15 : 1
P	75	10.7	7.62	0.61	10 : 1
SU		16.8	13.72	0.61	10 : 1
WB-12.0		25.9	18.20	0.61	15 : 1
WB-15.0		—	19.81	0.91	15 : 1
P	90	9.2	6.1	0.75	10 : 1
SU		15.2	12.19	0.61	10 : 1
WB-12.0		—	13.72	1.22	15 : 1
WB-15.0		—	18.29	1.22	15 : 1
P	105	—	6.1	0.76	8 : 1
SU		—	10.66	0.51	10 : 1
WB-12.0		—	12.19	1.22	10 : 1
WB-15.0		—	16.76	1.22	15 : 1
P	120	—	6.10	0.61	10 : 1
SU		—	9.14	0.91	10 : 1
WB-12.0		—	10.67	1.52	8 : 1
WB-15.0		—	13.72	1.22	15 : 1
P	135	—	6.10	0.45	15 : 1
SU		—	9.14	1.22	6 : 1
WB-12.0		—	9.14	2.44	6 : 1
WB-15.0		—	12.19	1.83	10 : 1
P	150	—	5.49	0.51	10 : 1
SU		—	9.14	1.22	8 : 1
WB-12.0		—	9.14	1.83	8 : 1
WB-15.0		—	10.67	2.14	6 : 1
P	180	—	4.57	0.15	20 : 1
SU		—	9.14	0.46	10 : 1
WB-12.0		—	6.00	2.89	5 : 1
WB-15.0		—	7.62	2.89	5 : 1

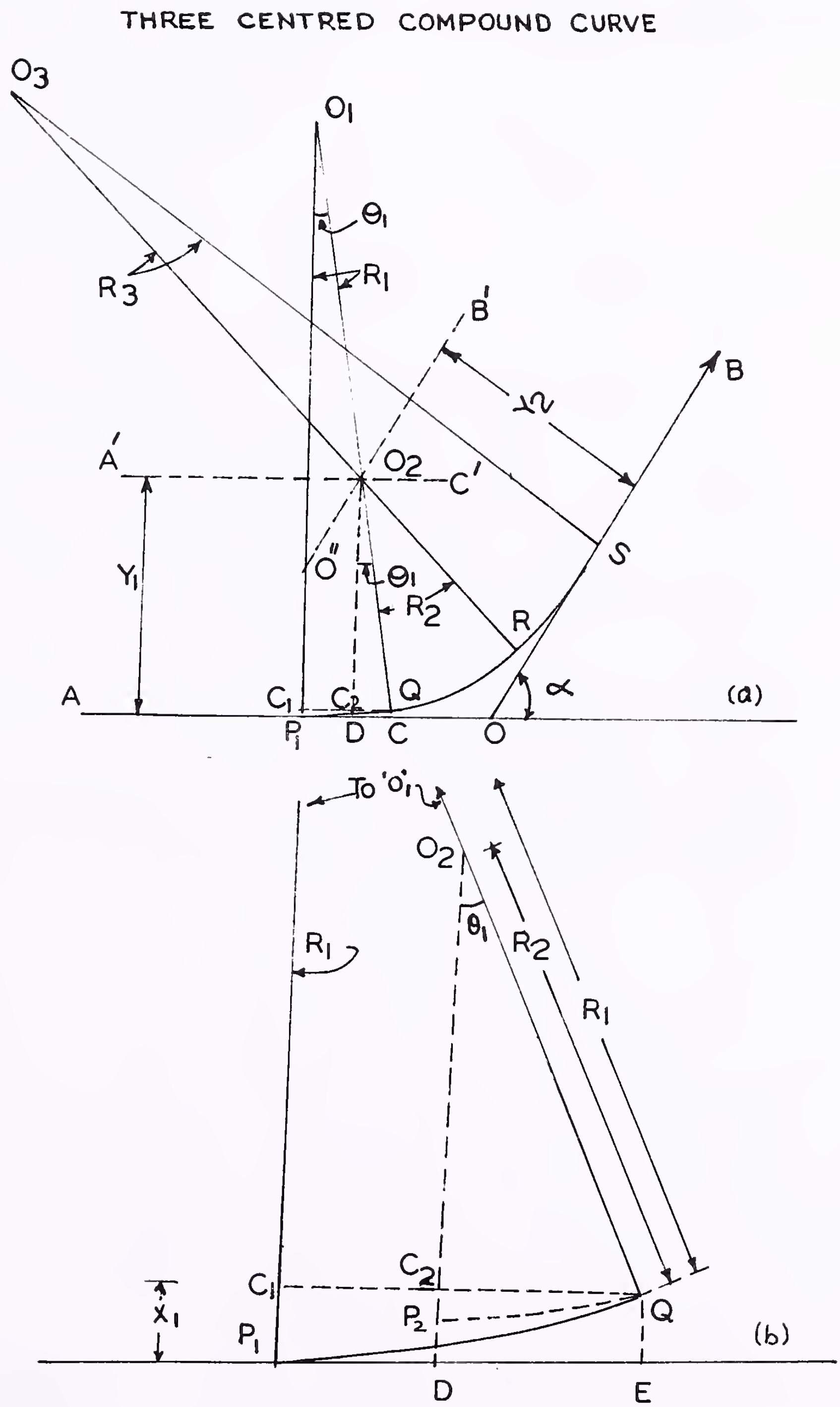


Fig. I-11. Three Centred Compound Curve

Similarly distance between O"B' and OB

$$y_2 = R_2 \left(1 - \frac{x_2}{R_3}\right) + x_2$$

$$= R_2 + x_2 - x_2 \frac{(R_2)}{R_3}$$

Procedure of Curve tracing :

Refer to Fig. I-11

1. Draw line AO and OB at an angle α = angle of turn.

2. Draw line A'O' and O' B' parallel to A O and O B at distance Y_1 and Y_2 respectively.

3. Knowing the Position of O_2 draw an arc of Radius R_2 with centre at O_2.

4. Draw line C_1 // A O at distance x_1 cutting the arc drawn with centre O_2 at Q.

5. Extend Q O_2 upto O_1 such that $O_1Q = R_1$ and with Radius R_1 and centre O_1 draw arc P_1 Q touching AO at P_1 (Tangent).

6. Similarly draw the other curves RS.

   P_1 Q R S is the required 3 centred curve with offset x_1 at Q and x_2 at R.

   Tables I-3, I-4 and I-5 give some typical values of Y_1, Y_2 and R_1, R_2 and R_3 for various angles of forms and design of vehicles depending upon whether the compound curve is symmetrical or assymmetrical with the help of these Tables, the 3-centred compound can be drawn.

Table I-3. 3-Centered Compound Curve (Symmetrical) Without Channelising Island

Design Vehicle	Angle of Turn	Radii (Metres)	Offset	Co-ordinates of Centre of Middle arc in metres
WB - 15.0	45°	61.0 - 30.5-61.0	0.91	30.955
WB - 15.0	60°	61.0-22.9-61.0	1.68	23.949
P SU WB 12.0 WB 15 - 0	75°	30.5-7.6-30.5 36.6-13.7-36.6 36.6-13.7-36.6 45.7-15.2-45.7	0.61 0.61 1.52 1.83	8.055 14.082 14.651 16.421
P SU WB - 12.0 WB - 15.0	90°	30.5-6.1-30.5 36.6-12.2-36.6 36.6-12.2-36.6 54.9-18.3-54.9	0.76 0.61 1.52 1.83	6.703 12.607 13.213 19.520
P SU WB - 12.0	105°	30.5-6.1-30.5 30.5-10.7-30.5 30.5-10.0-30.7	0.76 0.91 1.52	6.700 11.291 11.687
P SU WB - 12.0 WB - 15.0	120°	30.5-6.1-30.5 30.5-9.1-30.5 36.6-9.1-36.6 54.9-12.2-54.9	0.76 0.91 1.83 2.59	6.708 9.738 10.475 14.214
P SU WB - 12.0 WB - 15.0	135°	30.5-6.1-30.5 30.5-9.1-30.5 36.6-9.1-36.6 48.8-10.7-48.8	0.45 1.22 1.98 2.74	6.460 9.956 10.588 12.839
P SU WB-12.0 WB - 15.0	150°	20.9-5.9-20.0 30.5-9.1-30.5 30.5-9.1-30.5 48.8-10.7-48.8	0.76 1.22 1.83 2.13	6.445 9.955 10.384 12.363
P SU WB - 12.0 WB - 15.0	180° (U Turn)	15.2-4.6-15.2 30.5-9.1-30.5 30.5-6.1-39.6 39.6-7.6-39.6	0.15 1.37 2.90 2.90	4.705 10.061 8..420 9.943

Table I-4. 3 Centred Compound Curve (Assymetrical) Without Channelising Island

Design Vehicle	Angle of Turn	Radii Metres	Offset metre	Co-ordinates of centre of middle arc	
WB-150	60°	61.0-0.22.9-83.8	0.61-1.83	23.281	24.230
WB-12.0	75°	36.6-13.7-61.0	0.61-1.98	14.082	15.235
WB-15.0		45.7-15.2-68.6	0.61-3.05	15.607	17.574
WB-12.0	90°	36.6-12.2-61.0	0.61-1.83	12.607	13.664
WB-15.0		36.6-12.2-61.0	0.61-3.05	12.607	14.640
WB-12.0	105°	30.5-10.7-61.0	0.61-2.44	11.096	12.712
WB-15.0		45.57-12.2-64.0	0.61-3.05	12.647	14.669
WB-12.0	120°	30.5-9.1-54.9	0.61-2.74	9.523	9.609
WB-15.0		45.7-10.7-67.1	0.61-3.66	11.167	13.776
WB-12.0	135°	30.5-7.6-54.9	0.91-3.05	8.283	10.228
WB-15.0		39.6-9.1-56.4	0.91-4.27	9.301	12.681
WB-12.0	150°	27.4-7.6-48.8	0.91-3.35	8.258	10.428
WB-15.0		36.6-9.1-54.9	0.91-4.27	9.784	12.662
WB-12.0	180°	25.9-6.1-45.7	1.83-3.96	7.499	9.531
WB-15.0		30.5-7.6-54.9	1.83-3.96	8.974	11.012

Table I-5. 3 Centred Compound Curve (Symmetrical)-With Channelising Island

Angle of Turn	Radii Metres	Offset Metres	Co-ordinate of centre of middle arc (in metres)	
SU	75°	45.7-22.9-45.7	1.52	23.658
WB-15.0		54.9-27.4-54.9	1.07	27.936
SU	90°	45.7-15.2-45.7	1.52	16.214
WB-15.0		54.9-19.6-54.9	1.03	20.262
SU	105°	30.5-10.7-30.5	1.52	11.687
WB-15.0		54.9-13.7-54.9	2.44	15.531
SU	120°	30.5-9.1-30.5	0.91	9.738
WB-15.0		54.2-12.2-54.2	2.59	14.207
SU	135°	30.5-9.1-30.5	1.22	9.956
WB-15.0		48.8-10.7-48.8	2.74	12.839
SU	150°	30.5-9.1-30.5	1.22	9.956
WB-15.0		48.8-10.7-48.8	2.13	12.363

CAPACITY ASSESSMENT OF AT GRADE INTERSECTION (BASED ON U.K. PRACTICE)

1. Type of Movements & Capacity

1.1. The actual capacity of intersection depends on the type of vehicle manoeuvres, the distribution of gaps in the traffic stream and the drivers judgement in selecting gaps through which to execute the desired manoeuvre. There is only limited information available on the capacity of intersections in India, as such we have to fall back on information available abroad for guidelines, till relevant studies are completed in the country. Fig. II-1 gives the types of movements that occur at unsignalised intersections and the subsequent section describes the possible capacities for such movements.

(1) Merging : Fig. II - 1(a)

This movement may take place by means of an acceleration lane or simply by an entry curve.

An estimate of the maximum slip road flows for different main road flows for rural situations can be made by use of curves in Fig. II - 2 which shows these flows for different carriageway widths. There curves adopted in U.K. may not necessarily be applicable to all Indian traffic conditions. Where the expected slip road flows exceed the maximum flows given in the Fig. II-2 it will be necessary to provide an additional lane to the main road.

The slip road capacities do not apply when there are other nearby road connections and their effect is to increase or decrease the volume in the nearside lane according to their position and direction of flow. The effect is most marked by on-road connections upstream within 130 m and somewhat less at 150 m to 300 m distance.

For the effect of downstream off-road connections, reference should be made to weaving capacity limitations.

Where no acceleration lane is provided, the capacity -of the slip road entry onto a dual two-lane road may be assumed for design purposes to be 290 p.c.u's/hour less than with an acceleration lane, greater differences have sometimes been experienced.

(2) Diverging Fig. II-1(b)

The capacity of a slip road exit may be taken to be 1,200 p.c.u's/hour provided the deceleration lane is well-designed and sign-posting gives adequate warning well in advance of the junction. In the case of dual three or four-lane roads when the flow off the slip road approaches the capacity of one lane, it may be appropriate to reduce the main carriageway by one lane beyond the slip road connection.

(3) Cutting Fig. II-1 (c) & (d)

A cutting manoeuvre may be a simple right-turn cut of one stream traffic across the opposing stream, or across a two-way stream of traffic. These movements take place in the naturally occurring gaps in the traffic streams which are of suitable duration. The larger traffic gaps are utilised by several vehicles cutting at one time.

As in general, the flow of traffic on a road approximates to a random distribution, calculations can be made of 'Capacity' which depends on the size of gaps required and the volume and speed of the traffic stream which must be crossed. For conditions of good visibility the curves shown in Fig. II - 3 give the minor road flow which can cross different major road flows either one or two-way for different sizes of traffic gaps. These are applicable to rural intersections. For one-way flows, e.g. across one carriageway of a dual carriageway road it would be reasonable to assume average minimum gaps of 4 - 6 seconds, and across the dual carriageway 6-8 seconds. As per a study on its at grade-intersections for Indian mixed traffic conditions the critical gaps have been found to be of a similar magnitude. For the higher design speeds, larger gaps should be used. These capacities need to a single lane from the minor road ; capacity can be increased by providing two lanes if these could be used effecting.

(4) **Cutting and Merging :** Fig. II-1(e)

Unlike the simple cutting of two-way flows this manoeuvre requires traffic gaps of sufficient duration for the turning vehicle to accelerate to a suitable speed to join the far traffic stream and traffic gaps of 8-12 seconds may be needed ; Fig. II - 3 gives the appropriate volumes.

(5) **Compound Cutting and Merging :** Fig. II - 1(f)

This type of movements takes place at a simple 'T' intersection. The normal sequence is for the right turn (R_1) into the minor road to precede the right turn (R_2) from the minor road. Precise calculations are complex and for most purposes it is reasonably accurate to add half of R_1 to R_2 and the capacity can be calculated using the gap sizes appropriate for cutting and merging even in Section (4) above.

(6) **Reservoir Space :** Fig. II-1 (g)

Where vehicles wait for opportunities to cross traffic streams, queues of vehicles develop and these need to be accommodated in suitable 'reservoir' spaces. The number of vehicle spaces needed varies due to the chance arrival of the turning vehicles and also due to the distribution of gaps of sufficient duration for the crossing to be made.

For roads with deceleration lanes in the central reserve it is not usually necessary to consider added length to the deceleration lane to cater for the wide variation in the number of vehicle waiting to turn.

For some intersection designs, such as left/right staggered intersections, the storage area is limited. These intersections should preferably be used up to the limit of capacity found by using 8 second gaps, and a storage space should be allowed for 8 vehicles. For lighter flows a minimum of 4 vehicle spaces is recommended. A length of 6 m for each p.c.u. may be assumed.

(7) **Weaving :** Fig. II - 1 (h) and (i)

The term weaving is applied to the combined movement of merging and diverging. There are two distinct types, namely (a) weaving at round abouts and (b) weaving along roads. In both cases there are normally other traffic streams which are non weaving, see Fig. II - 1(h) & II - 1(i) which indicates weaving streams and non-weaving streams.

1.2. In the case of roundabouts the capacity can be estimated from the capacity formula adopted in IRC: 65 - 1976.

2. Capacity of Intersections in Urban Conditions

2.1. An assessment of the intersection capacity in urban conditions can be made from the curves developed for the conditions and given in Fig. II -4. These curves are applicable to both T-intersections and cross roads. Staggered cross roads should be regarded as two T-intersections for direct cross roads the curves indicate the traffic that can enter from the more heavily trafficked side road.

2.2. The curve A in Fig. II - 4 shows the maximum volume of side road traffic which may cross different through flows on the main road based on the assumptions of a single lane side approach per visibility from the side road and a minimum acceptable gap of 8 seconds. The poor visibility will imply that when visibility on the approach road is restricted but is adequate for stop conditions.

2.3. Curve B in Fig. II - 4 shows the increased capacity under conditions of good visibility thus permitting vehicles from the side road to enter suitably in long gaps in platoons without each vehicle necessarily having to stop at the major road is based on a minimum acceptable gap of 6 seconds.

3. Weaving along roads : Fig. II-1(i)

The effect of weaving along roads on road capacity depends upon the spacing of interesections and volumes of weaving traffic streams. To calculate the appropriate road width and spacing of intersections reference should be made to Fig. II-1(i).

The number of lanes required for minimum weaving sections may be estimated from the formula given below. To allow for mixed traffic, mainly flows should be given in PCU's factors applicable to urban area.

$$N = \frac{W_1 + 3W_2 + F_1 + F_2}{C}$$

Where

N	=	Number of lanes
W_1	=	larger weaving flow in PCUs/hr.
W_2	=	smaller weaving flow in PCUs/hr.
F_1 & F_2	=	outer non-weaving stream in PCUs/hr.
C	=	normal lane capacity of the major road.

For given total volumes of weaving traffic, the minimum length of weaving section can be read from the diagram and the lane capacity for the minor weaving movement can be determined from right hand scale (Fig. II-5).

The number of lanes required in the weaving section can be found by summing the total of the non-weaving flow plus the major weaving flow and dividing by 1,200 and adding to this the minor weaving flow divided by the appropriate lane capacity for minor weaving movement. The latter can be obtained from Fig. II-5 given the weaving length and total weaving volume, see example given in the diagram. Where the calculated width is less than 3 lanes and one of the non-weaving flow exceeds 600 p.c.u.'s/hour, two additional lanes should be provided for non-weaving traffic.

4. Signal Controlled Movements (See Chapter 7 also)

In some instances particularly in semi-rural areas where signal control is applicable intersection may be designed on channelising principles and signal control applied to simple traffic counts. In such instances capacity checks could be made by taking the maximum flows per lane width (expressed in p.c.u's) for each phase and summed for the two phases. This sum then can be compared with a practical capacity of 1,600 p.c.u's/hour per traffic lane, thus lane densities in two phases of 800/800, 1,000/600, 1,200/400, etc. would be maximum practical capacities according to the proportions in the two phases.

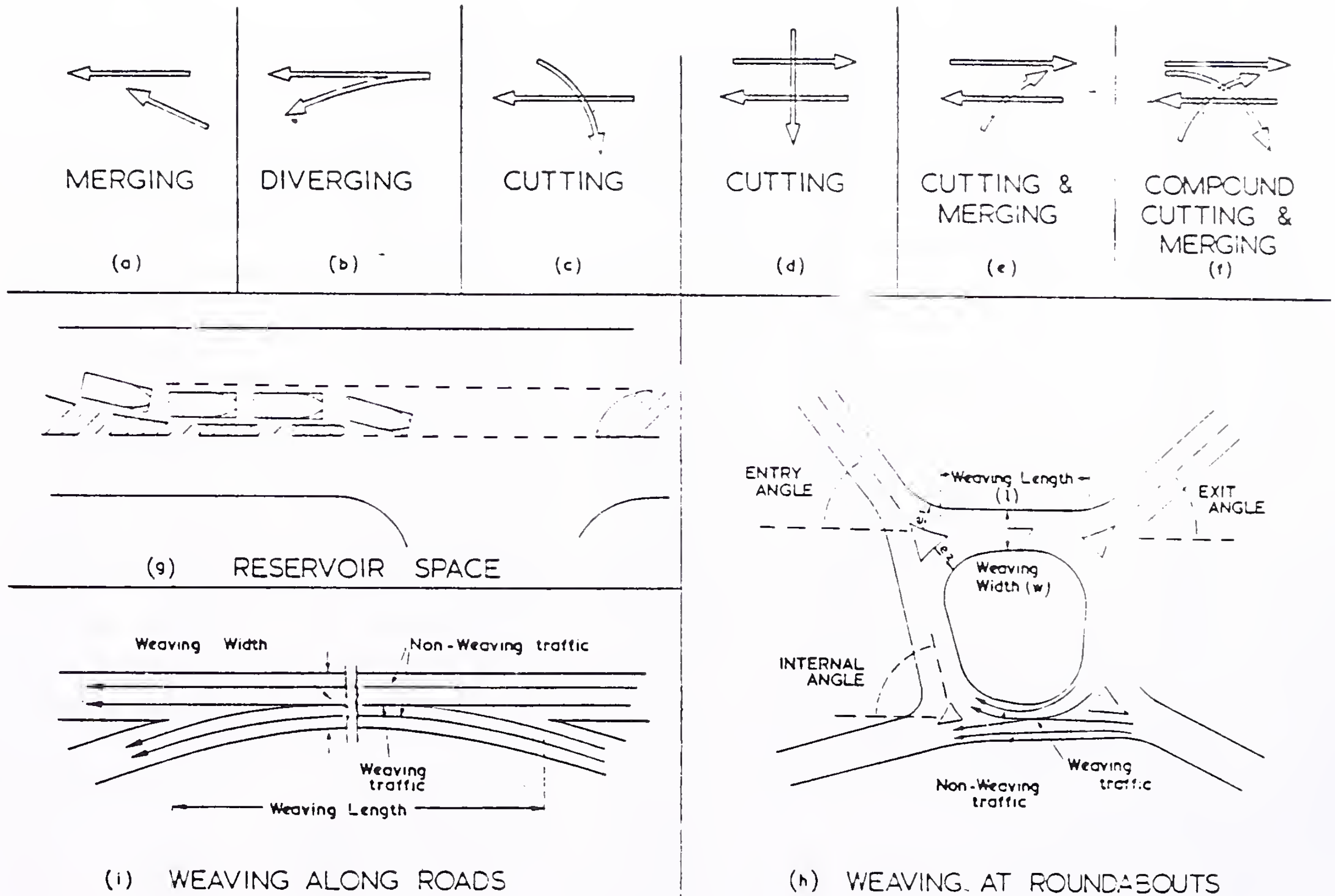


Fig. II-1. Uncontrolled Movements

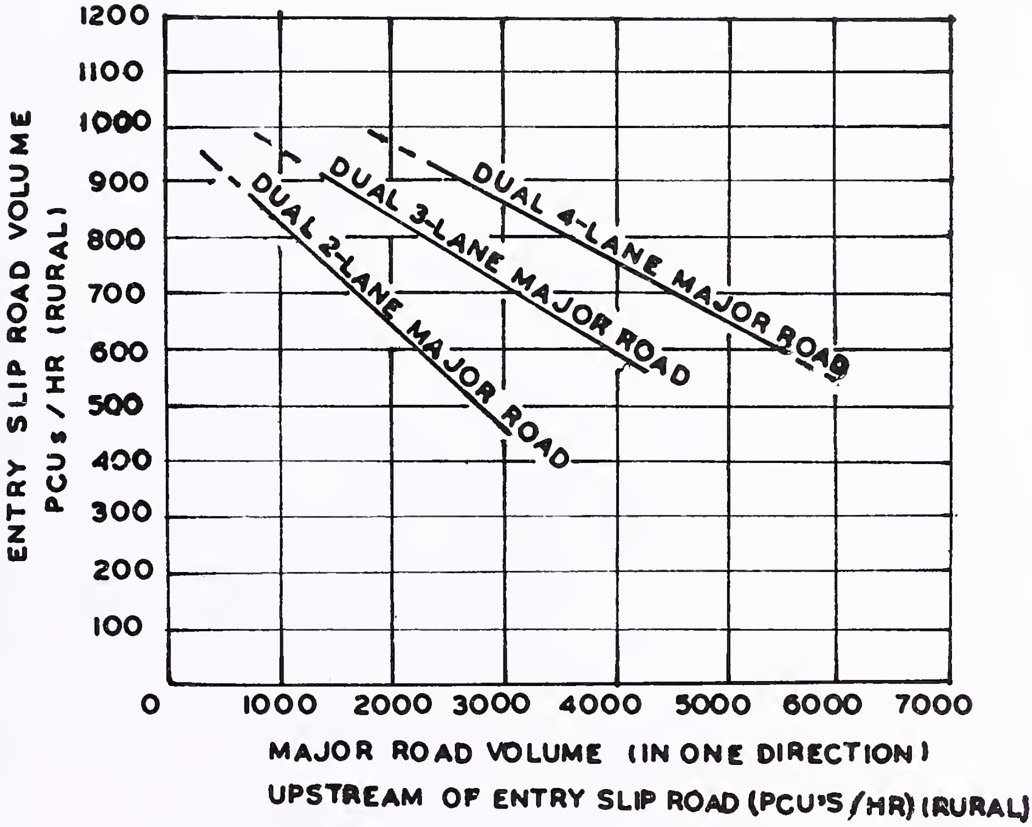


Fig. II-2. Capacity of Merging Flows

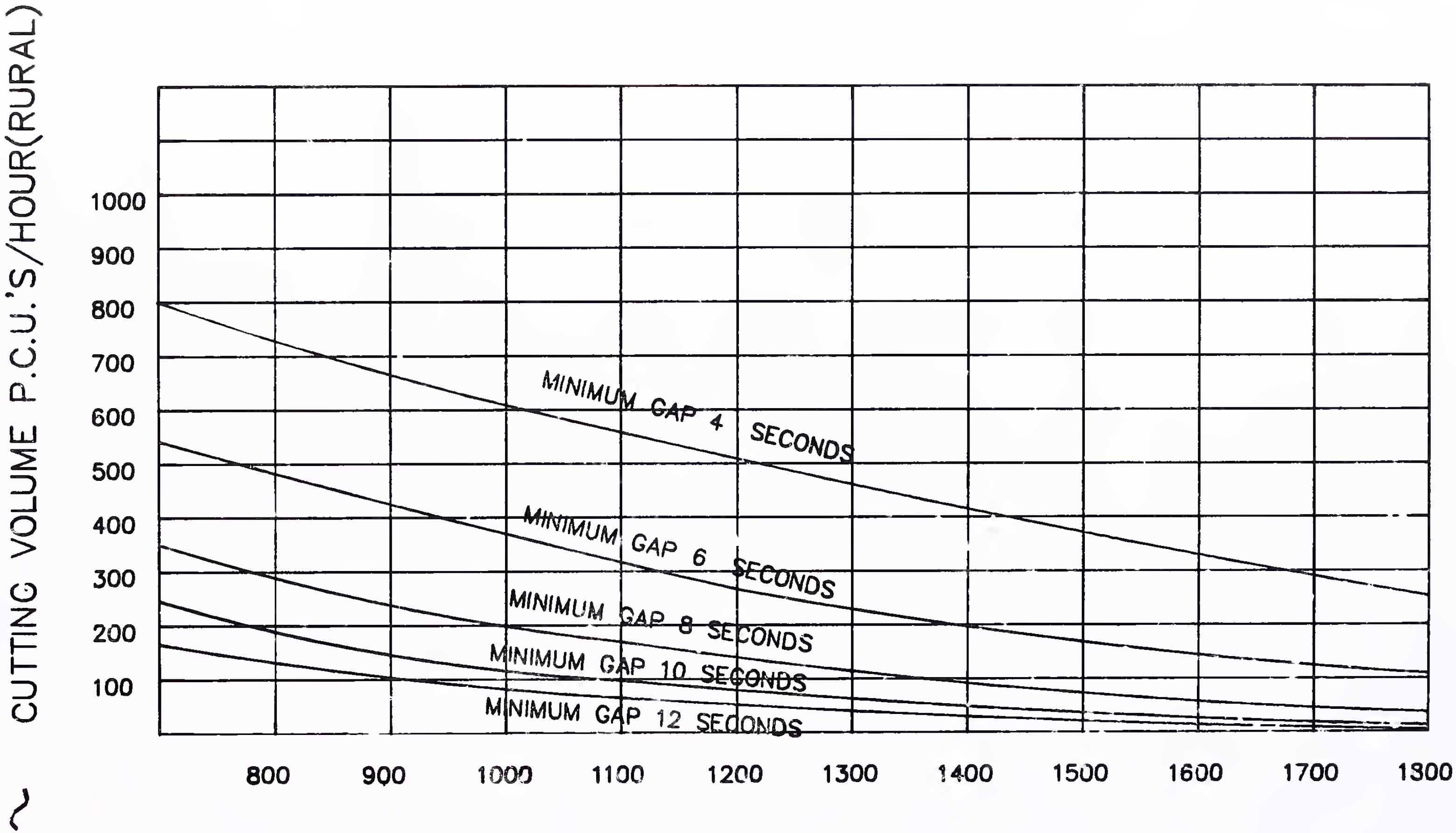


Fig. II-3. Volume of Traffic being cut p.c.u's/hour (Rural)

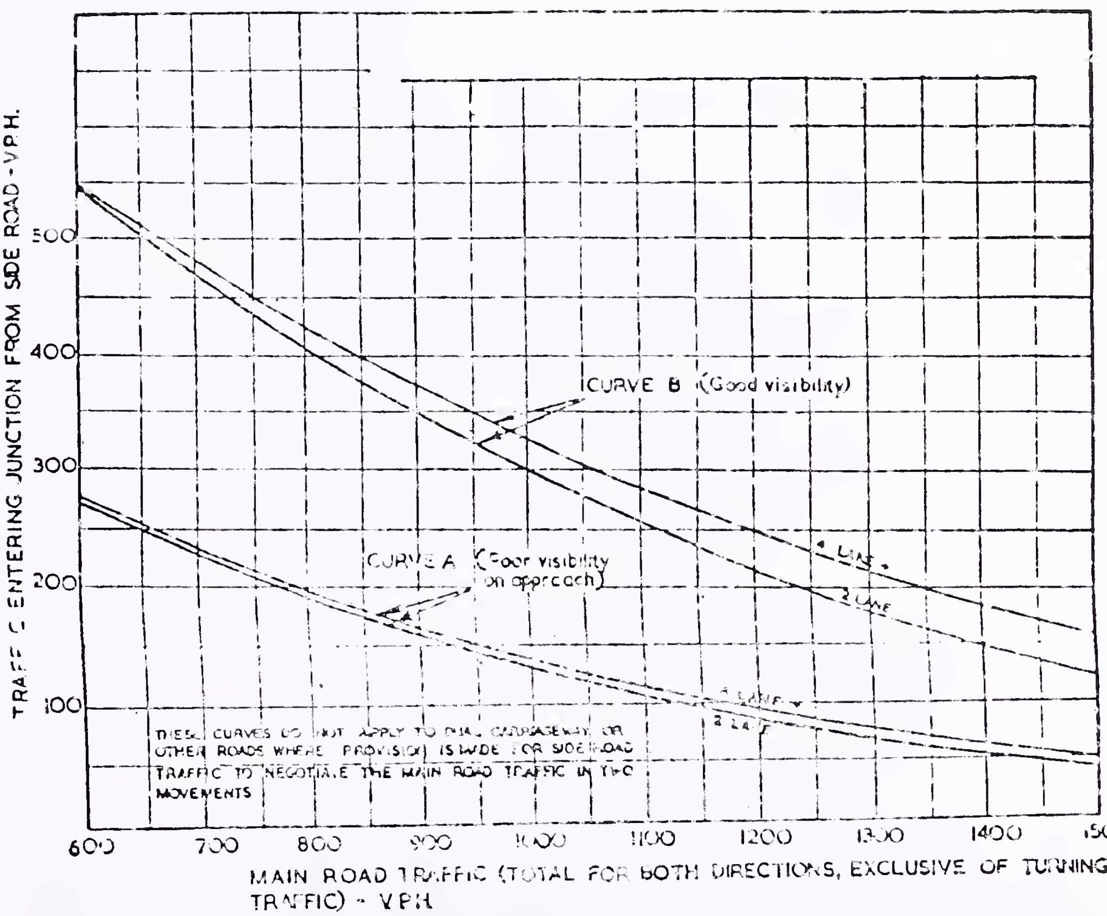
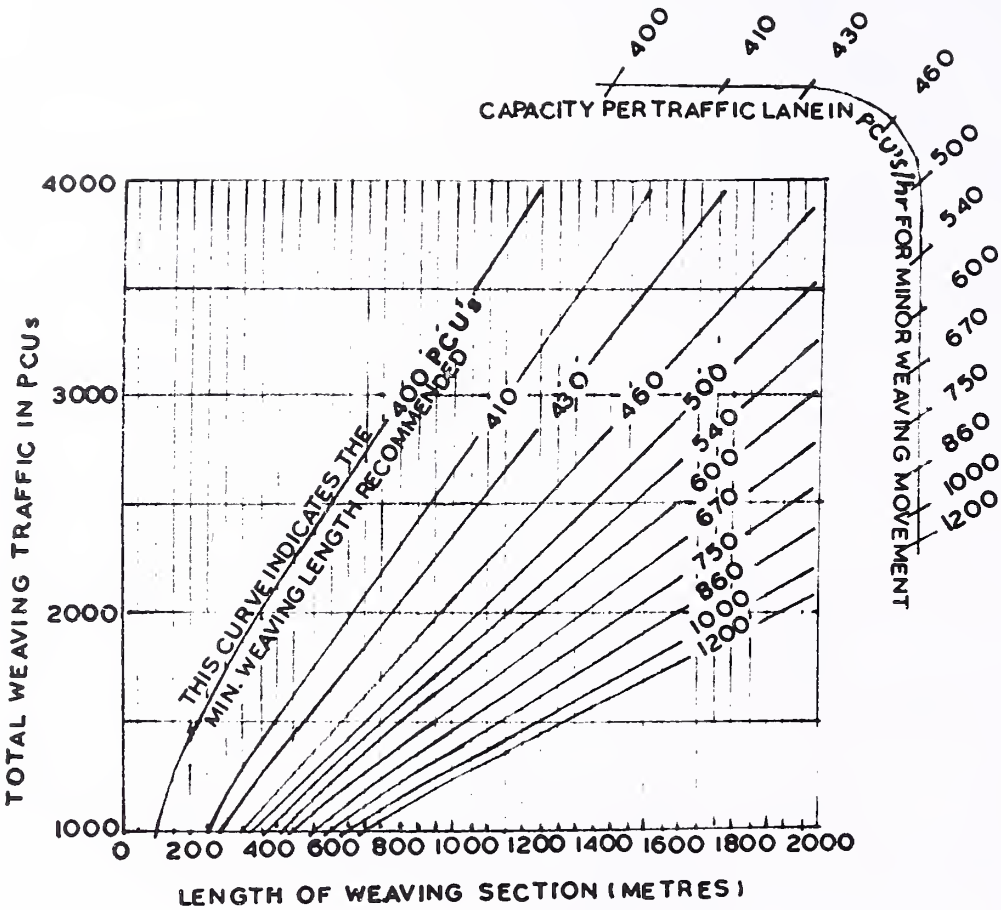


Fig. II-4. Capacity of uncontrolled Intersection



EXAMPLE :- LENGTH BETWEEN JUNCTIONS - 1600 M
NON WEAVING TRAFFIC — 1700 pcu's/hr,
MAJOR WEAVING TRAFFIC — 1450 pcu's/hr.
MINOR WEAVING TRAFFIC — 550 pcu's/hr.

FROM DIAGRAM FOR TOTAL WEAVING VOLUME OF 2000 pcu's / hr. AND WEAVING LENGTH 1600 m. CAPACITY PER LANE FOR MINOR WEAVING MOVEMENT = 880 pcu's / hr PER LANE.

∴ NUMBER OF LANES IN WEAVING SECTION IS —

$$\frac{1700+1450}{1200} + \frac{550}{880} = 3.25$$

i.e 4 TRAFFIC LANES

NOTE : PCU'S GIVEN ABOVE ARE RURAL STANDARD BASED ON MATERIAL THE COPYRIGHT OF THE HIGHWAY RESEARCH BOARD, U.S.A.

Fig. II-5. Capacity of Long Weaving Section Operating through Speed 70 km/hr.

CAPACITY OF UNSIGNALISED INTERSECION

1.1. Introduction

The analysis of unsignalized intersections is generally applied to existing locations either to evaluate existing operational conditions under present demands,or to estimate the impacts of anticipated new demands. The methodology is specifically structured to yield a level of service (LOS) and an estimate of reserve capacity for an existing case. Thus, operational analysis is the mode in which it is used. Design applications are treated as trial-and-error computations based on anticipated improvements to an existing intersection or on the design of a new intersection for the projected traffic.

The method generally assumes that major street traffic is not affected by minor street flows. The methodology also adjusts for the additional impedence of minor street flows on each other and accounts for the share use of lanes by two or three minor street movements.

1.2. Field Data Requirements

Computations require following types of data inputs to the methodology :

(i) Volume by type of movement for the design hour
(ii) Vehicle classification for the design hour
(iii) Peak hour factor (if peak flow rates are being used as the basis for analysis)
(iv) Prevailing (average running) speed of traffic on the major street
(v) Number of lanes on the major street
(vi) Number of lanes on the minor street approaches
(vii) Other geometric features i.e. channelization, angle of intersection, sight distance, corner radii, acceleration lanes, etc.
(viii) Type of control on the minor approaches.

Because the methodology herein results in a qualitative evaluation of delay, it is also recommended, if possible, that some delay data be collected with the above information. This will allow for a better quantification and description of existing operating conditions at the location under study. It would also allow for a more precise comparison with a signalized intersection analysis for which delay estimates are generated.

1.3. Sequence of Computations

As the methodology is based on a prioritized use of gaps by vehicles at an unsignalized intersection, it is important that computations be made in a precise order. The computational sequence is the same as the priority of gap use, and movements are considered in the order i.e. Left turns from the minor street, Right turns from the major street, through movements from the minor street and Right turns from the minor street.

1.4. Analysis of Four-Leg Intersections

The following steps describe the procedure of computations :

1.4.1. Volume summary and adjustment : The first page of the worksheet [Fig. III (5a)] consists of summarization and adjustment of demand volumes. Basic geometric data are also summarized on this page.

(i) Hourly volumes are summarized on the top portion of the form on the diagram provided. A 'north' indication should be inserted to ensure proper orientation of the intersection and of the demand volume. V_1 to V_6 denote movements and on major street and V_7 to V_{12} denote movements on minor street.

(ii) The number of lanes on each approach should be indicated.

(iii) The type of control is indicated by checking the appropriate box, and the prevailing speed on the major street and the Peak Hourly Flow (PHF) be listed.

(iv) Volume adjustments are made to convert Volume Per Hour (VPH) to Passenger Car Per Hour (PCPH). In general, analysis will be on the basis of full hour volumes. The volumes of all categories of vehicles have to be converted into PCPH. Through and left turning volumes on major street would not be converted to PCPH as they are only utilized in computation of 'Conflicting Traffic Volumes' which is done in terms of VPH.

(v) The conversion from VPH to PCPH is made using the passenger car equivalent values as given in Table 1. Also, find the total volume (PcPh) for all categories.

Table III -1. PCU Values for Rural/Urban conditions

(i)	Passenger car, tempo, auto-rickshaw and tractor (without trailer)	1.00
(ii)	Cycle, Motor/Scooter,	0.50
(iii)	Lorry, Bus and Tractor-trailer unit	3.00
(iv)	Cycle-Rickshaw	1.50
(v)	Horse Driven vehicle	4.00
(vi)	Bullock carts (big)	8.00
(vii)	Bullock carts (small)	6.00

1.4.2. Computation of Movement Capacities : The second page of the worksheet is for the computation of movement capacities for each subject movement. Equations given on this worksheet can be used for computation by taking the data of volumes as shown in first page of worksheet. The volume denoted as V (v) referes to volumes in VPH. Computations proceed in a prescribed order. Considering first the left turns from the minor stret followed by right turns from major street, through movements from minor street and right turns from minor street.

For each movement, the following sequence of computations is followed :

(i) Using Figure III-1, compute conflicting flows, V

(ii) Find the critical gap, T in sec (From Table III - 2) and adjustments, if required. For each type of control on minor road the value of critical gap would depend on the average running speed on major street and on whether it is a 2-lanes or a 4-lane street.

(iii) Find the potential capacity, C in PCPH (From Figure-III-2) based on conflicting flows computed in (i) above

(iv) Compute the per cent of potential capacity used by the movement.

(v) Find the impedance factor, P using Figure III-4 based on the percentage of capacity used by existing demand as computed in step (iv) above. This factor will be used to adjust the capacity of lower priority movement for impedance. The impeded movements and their capacity are shown in Fig. III-3.

(vi) Compute the movement capacity based on impedence computations given in Fig. III-3. Since no movements impede left turns from major street (basic assumption that major street traffic is not impeded by minor street flows). Their capacity is same as the potential capacity for these movements. (Steps 1 & 2 of Fig. III-5(b).

1.4.3. Computation of shared-lane Capacity and Level of Service : The third page of the worksheet is used to compute shared lane capacities, reserve capacities, and level of service. The user will have to determine from field data or available design plans, the details of movements that share a lane. The appropriate computations for shared-lane capacity are made using the equations shown on the worksheet depending upon the number of movements (2 or 3) sharing a lane. Reserve capacity is then computed for each lane. The level of service is determined from Table III-3.

1.5. Analysis of T-Intersections

The analysis of T-intersections follows the same general steps as those described above for four leg intersections. They are, however very much simplified, because many of the movements and the majority of the conflicts present in a four leg intersection are removed. Because of this, a simplified worksheet is provided for T-intersection computations, and is shown in Fig. III-6.

1.6. Sample Calculations - A Case Study

1.6.1. Computation of capacity and LOS for 4-leg intersection : A 4-leg intersection has been analysed for finding out the capacity. The details of the intersections is shown in Fig. 5(a). This intersection formed by the joining of National Highway NH-24 with a Major District Road (MDR) at km 86.00 on NH-24 (Delhi-Lucknow Road). The classified volume data has been collected for one hour duration in vehicle per hour (VPH). The above volume has been converted into passenger car per hour (PCPH) for each approach using the passenger car equivalent value as given in Table III-1. The volume in PCPH has been shown for each arm and for each movement of the calculation for capacity and level of service is shown in Fig. III-5(b).

1.6.2. Computation of capacity and LOS for 3 leg intersection : A 3-leg intersection has been analysed for finding out the capacity. The details of the intersection and calculation in the form of worksheet is shown in Fig. III-6. This intersection is formed by joining NH-24 to a link road at km 20 on NH-24. The classified volume has been collected for one hour & converted upto PCPH for each approach. The volume in PCPH has been shown for each arm and for each movement of traffic. The steps of the calculation for capacity and level of service is shown in Fig. III-6.

Table III-2. Basic Critical Gap (in Secs.) For Passenger Cars

Vehicle Manoeuver and type of Control	Average runing speed, major road (kmph)																			
	16		24		32		40		48		56		64		72		80		88	
No. of Lanes in Major Road	2	4	2	4	2	4	2	4	2	4	2	4	2	4	2	4	2	4	2	4
LT from Minor Road																				
Stop	4.7	4.7	4.9	4.9	5.1	5.1	5.3	5.3	5.5	5.5	5.7	5.7	5.9	5.9	6.1	6.1	6.3	6.3	6.5	6.5
Yield	4.6	4.6	4.7	4.7	4.8	4.8	4.9	4.9	5.0	5.0	5.1	5.1	5.2	5.2	5.3	5.3	5.4	5.4	5.5	5.5
RT from Major Road	4.6	5.1	4.7	5.2	4.8	5.3	4.9	5.4	5.0	5.5	5.1	5.6	5.2	5.7	5.3	5.5	5.4	5.9	5.5	6.0
Cross Major Road																				
Stop	4.8	5.3	5.1	5.6	5.4	5.9	5.7	6.2	6.0	6.5	6.3	6.8	6.6	7.1	6.9	7.4	7.2	7.7	7.5	8.0
Yield	4.7	5.2	4.9	5.4	5.1	5.6	5.3	5.8	5.5	6.0	5.7	6.2	5.9	6.4	6.1	6.6	6.3	6.8	6.5	7.0
RT from Major Road																				
Stop	5.3	5.8	5.6	6.1	5.9	6.4	6.2	6.7	6.5	7.0	6.8	7.3	7.1	7.6	7.4	7.9	7.7	8.2	8.0	8.5
Yield	5.2	5.7	5.4	5.9	5.6	6.1	5.8	6.3	6.0	6.5	6.2	6.7	6.4	6.9	6.6	7.1	6.8	7.3	7.0	7.5

Adjustments and Modifications to Critical Gap, Sec

	Conditions	Adjustments		Remarks
(i)	LT from Minor Street (Curb radius > 15m, turn angle < 60°)	- 0.5	- -	Maximum total decrease in critical gap = 1.0 sec. Max. Critical gap = 8.5 sec
(ii)	LT from Minor Street (Acceleration lane provided)	- 1.0	-	Interpolation can be made for values of average running speed between 48 and 88 kmph.
(iii)	All Movement (Population > 2.5 lacs)	- 0.5	*	The adjustment is made for the specific movement if impacted by restricted sight distance
(iv)	Restricted Sight Distance	+ 1.0		

TABLE III-3 : Level-of-Service Criteria for Unsignalized Intersections

Reserve Capacity (pcph)	Level of Service (LOS)	Expected Delay to Minor Street Traffic
> 400	A	Little or no delay.
399 - 300	B	Short traffic delays
299 - 200	C	Average traffic delays
199 - 100	D	Long traffic delays
99 - 0	E	Very long traffic delays
-	F	Stops-and-starts

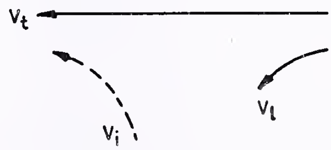
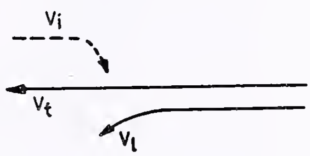
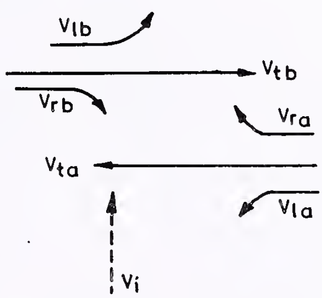
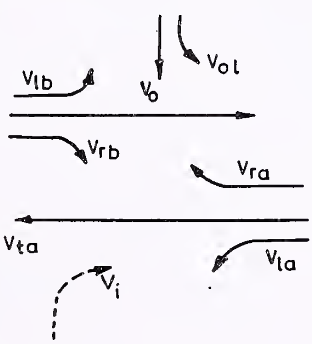
ILLUSTRATION	CONFLICTING TRAFFIC, V_{ci}	SUBJECT MOVEMENT
1 	$\frac{1}{2}(V_l) + V_t$	LEFT TURN FROM MINOR STREET
2 	$V_l + V_t$	RIGHT TURN FROM MAJOR STREET
3 	$\frac{1}{2}(V_{la}) + V_{ta} + V_{ra}$ $+ V_{lb} + V_{tb} + V_{rb}$	THROUGH MOVEMENT FROM MINOR STREET
4 	$\frac{1}{2}(V_{la}) + V_{ta} + V_{ra}$ $+ V_{lb} + V_{tb} + V_{rb}$ $+ V_o + V_{ol}$	RIGHT TURN FROM MINOR STREET

Fig. III-1. Definition and Computation of Conflicting Traffic Volumes

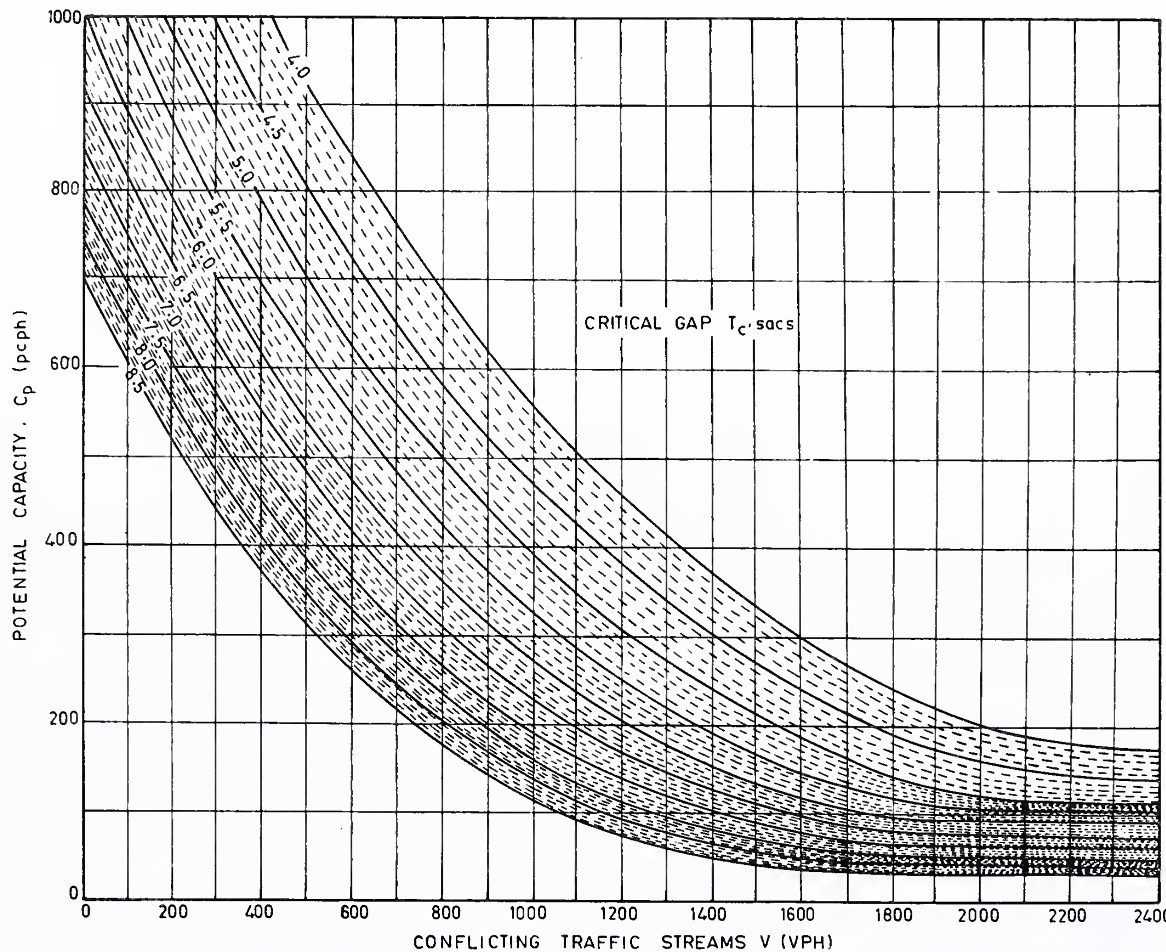


Fig. III-2. Potential Capacity Based on Conflicting Traffic Volume and Critical Gap Size

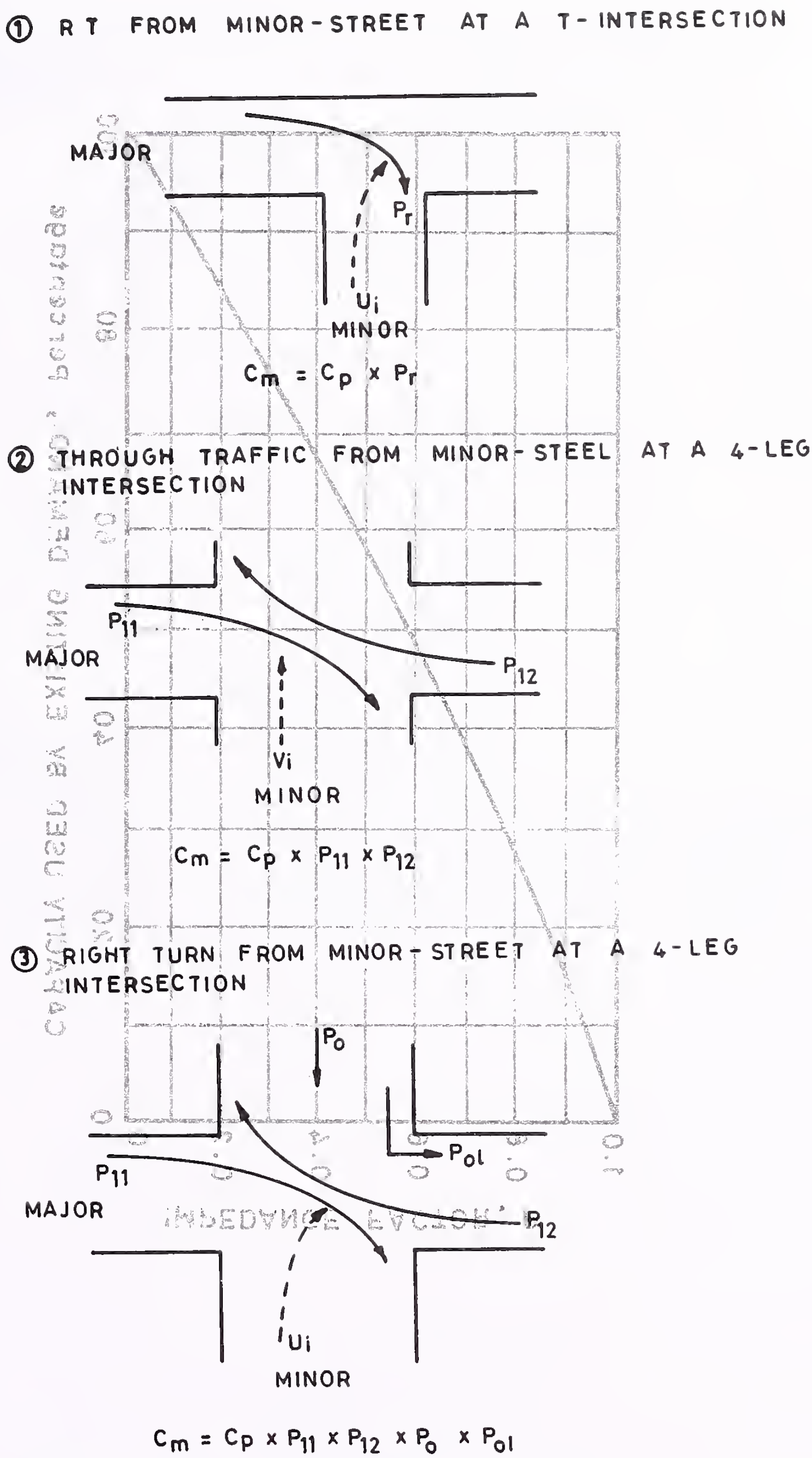


Fig. III-3. Illustration of Impedance Computations

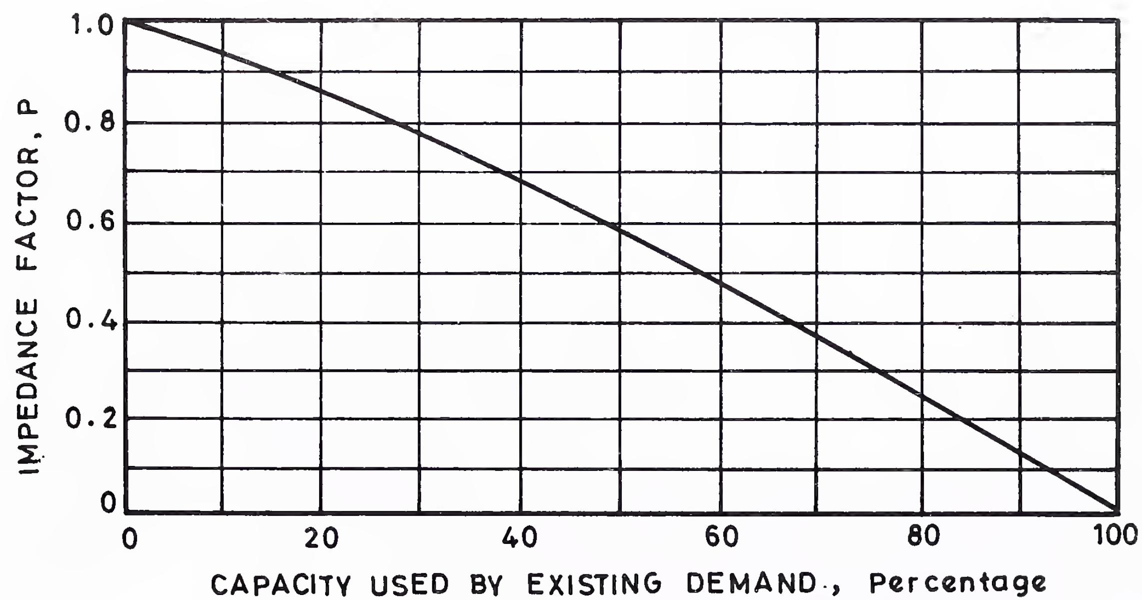
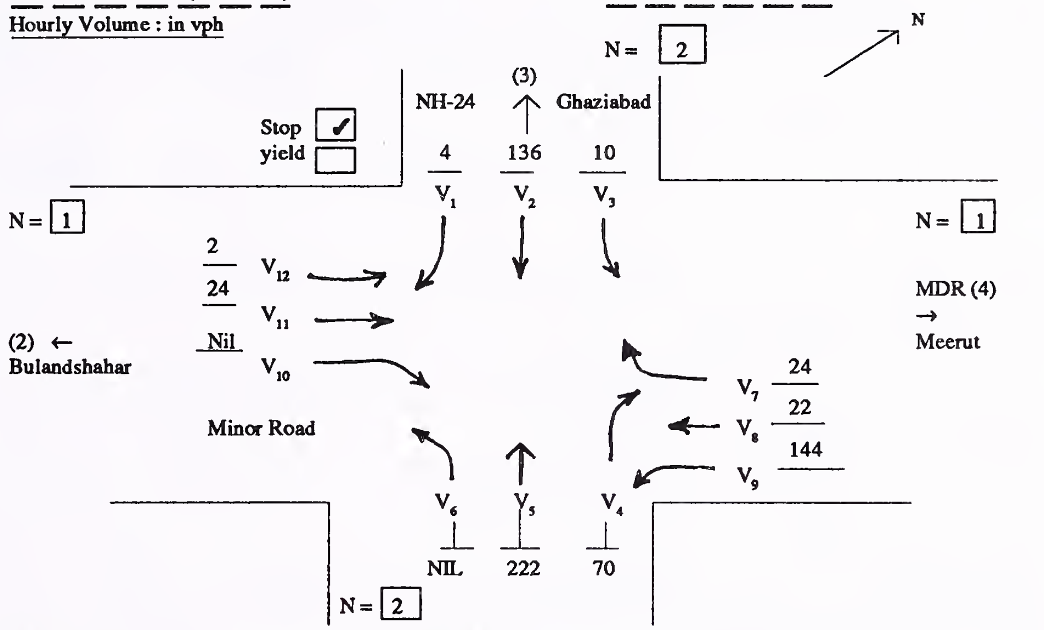


Fig. III-4. Impedance Factors as a Result of Congested Movements

Name : KM 86/000 (NH/MDR)

Laction : Garhmukteshwar

Hourly Volume : in vph



Date of Counts

Time Periods

Average Running Speed 32 kmph

Major road ↓ Moradabad (1)

												Volume Adjustment
1	2	3	4	5	6	7	8	9	10	11	12	Movement No.
4	136	10	70	222	Nil	24	22	144	Nil	24	2	Volume (vph)
4	163	12	85	265	Nil	30	27	173	NIl	30	2	Volume (pcph)

Volume in pcph

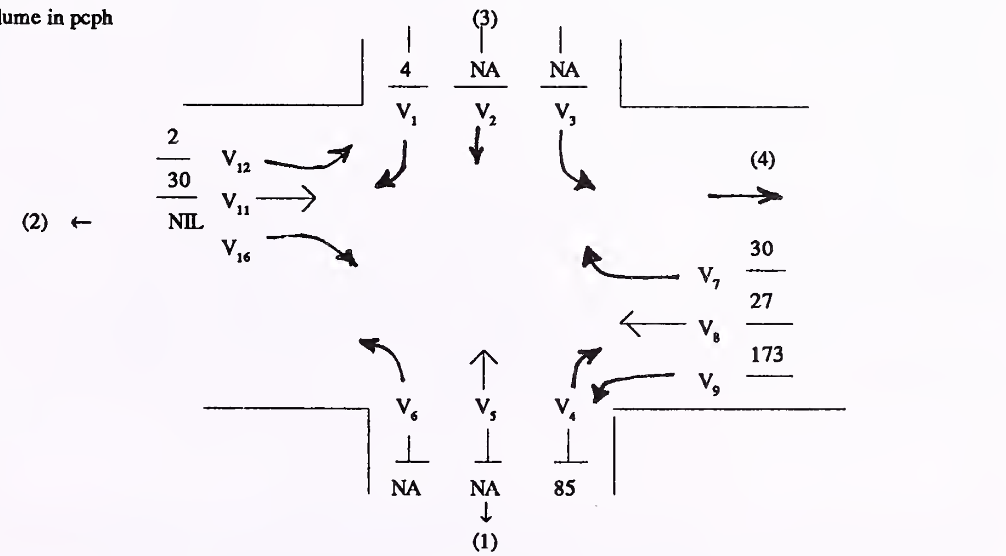


Fig. III 5(a). Worksheet for Four Leg Intersections

STEP 1 : LP FROM MINOR STREET	(V_9)	(V_{12})
Conflicting Flows Vc	$1/2\ V_3 + V_2 + V_{c9}$ $1/2 \times 10 + 136 = 141$ vph	$1/2\ V_6 + V_5 = V_{c12}$ $- + 222 = 222$ vph
Critical Gap. Tc (Table III-2)	5.1 (sec)	5.1 (sec)
Potential Capcity C_p (Fig. III-2.)	C_{p9} 1000 pcph	$C_{p12} = 9.10$ pcph
Per cent of C_p Utilized	$(V_9/C_{p9}) \times 100 = \dfrac{173 \times 100\ \%}{1000}$ $= 17.3\%$	$(V_{12}/C_{p12}) \times 100 = \dfrac{2 \times 100\ \%}{940}$ $= 0.21\%$
Impedance Factor. P (Fig. III-4)	$P_9 = 0.92$	$P_{12} = 0.99$
Actual Capaciy. Cm	$Cm_9 = C_{p9} = 1000$ pcph	$Cm_{12} = C_{p12} = 940$ pcph

STEP 2 : RT FROM MAJOR STREET	(V_4)	(V_1)
Conflicting Flows. Vc	$V_3 + V_2 + Vc_4$ $10 + 136 = 146$ vph	$V_6 + V_5 = V_{CL}$ $- + 222 = 222$ vph
Critical Gap. Tc (Table III-2)	4.8 (sec)	4.8 (sec)
Potential Capcity C_p (Fig. III-2.)	$C_{P4} =$ 1000 pcph	$C_{p1} = 1000$ pcph
Per cent of C_p Utilized	$(V_4/C_{p4}) \times 100 = \dfrac{85 \times 100}{1000}$ $= 8.5\%$	$(V_1/C_{p1}) \times 100 = \dfrac{4 \times 100\ \%}{1000}$ $= 0.4\%$
Impedance Factor P (Fig. III-4)	$P_4 = 0.96$	$P_1 = 0.98$
Actual Capaciy. Cm	$C_{m4} = C_{p4} = 1000$ pcph	$C_{m1} = C_{p1} = 1000$ pcph

STEP 3 : TH FROM MINOR STREET	(← V_8)	(→ V_{11})
Conflicting Flows. Vc	$1/2\ V_3 + V_2 + V_1 + V_6 + V_5 + V_4 = V_{c8}$ $1 \times 10 + 136 + 4 + - + 222 + 70$ $= 437$ vph	$1/2\ V_6 + V_5 + V_4 + V_3 + V_2 = V_1 = V_{c11}$ $- + 222 + 70 + 10 + 136 + 4 = 442$ vph
Critical Gap. Tc (Table III-2)	5.4 (sec)	5.4 (sec)
Potential Capcity C_p (Fig. III-2)	$C_{P8} = 680$ pcph	$C_{p11} = 670$ pcph
Per cent of C_p Utilized	$(V_8/C_{p8}) \times 100 = \dfrac{27 \times 100}{680}$ $= 3.97\%$	$(V_{11}/C_{p11}) \times 100 = \dfrac{30 \times 100\ \%}{670}$ $= 4.47\%$
Impedance Factor. P (Fig. III-4)	$P_5 = 0.98$	$P_{11} = 0.98$
Actual Capaciy. Cm	$C_{m8} = C_{p8} \times P_1 \times P_4$ $680 \times .98 \times .96 = 640$ pcph	$C_{m11} = C_{p11} \times P_1 \times P_4 =$ $670 \times 0.98 \times 0.96 = 631$ pcph

STEP 4 : RT FROM MINOR STREET	(V_7)	(V_{10})
Conflicting Flows Vc	Vc_3 (step 3) $+ V_{11} + V_{12} = V_{c7}$ $437 + 24 + 2 = 463$ vph	V_{c11} (step) $+ V_5 + V_9 = V_{c10}$ $442 + 22 + 144 = 608$ vph
Critical Gap. Tc (Table III-2)	5.8 (sec)	5.8 (sec)
Potential Capcity . C_p (Fig. III-2)	$C_{P7} =$ 585 pcph	$C_{p10} =$ 470 pcph
Per cent of C_p Utilized	$C_{m7} = Cp_7 \times P_1 \times P_4 \times P_{11} \times P_{12}$ $585 \times .98 \times 0.96 \times .98$ $\times .99$	$C_{m10} = Cp_{10} \times P_4 \times P_1 \times P_5 \times P_9$ $= 470 \times 0.96 \times .98$ $\times 0.98 \times 0.92$
Actual Capaciy. Cm	= 534 pcph	= 399 pcph

Fig III 5(b). Worksheet for Four Leg Intersections

WORKSHEET FOR FOUR LEG INTERSECTIONS

SHARED LANE CAPACITY

$$C_{SH} = \frac{V_1 + V_2}{(V_i/C_{mi}) + (V_j/C_{mj})}$$ where 2 movements share a lane

$$C_{SH} = \frac{V_i + V_j + V_k}{/C_{mi}) + (V_j/C_{mj}) + (V_k/C_{mk})}$$ where 3 movements share a lane

MINOR STREET APPROACH MOVEMENTS 7, 8, 9

Movement	V(pcph)	Cm(pcph)		C_{SH}(pcph)		C_R = Csh - V		LOS
7	30	534 *		534 *		504		A
8	27	640	848+	640	618*	613	A	A
9	173	1000		1000		827		A

MINOR STREET APPROACH MOVEMENTS 10, 11, 12

Movement	V(pcph)	Cm(pcph)		C_{SH}(pcph)		C_R = Csh - V		LOS
10	NIL	399		0		399		B
11	30	631	640	631	608	601	A	A
12	2	940		940		938		A

MAJOR STREET RIGHT TURNS 1, 4

Movement	V(pcph)	Cm(pcph)	$C_R = C_m$ - V	LOS
1	4	1000	996	A
4	85	1000	915	A

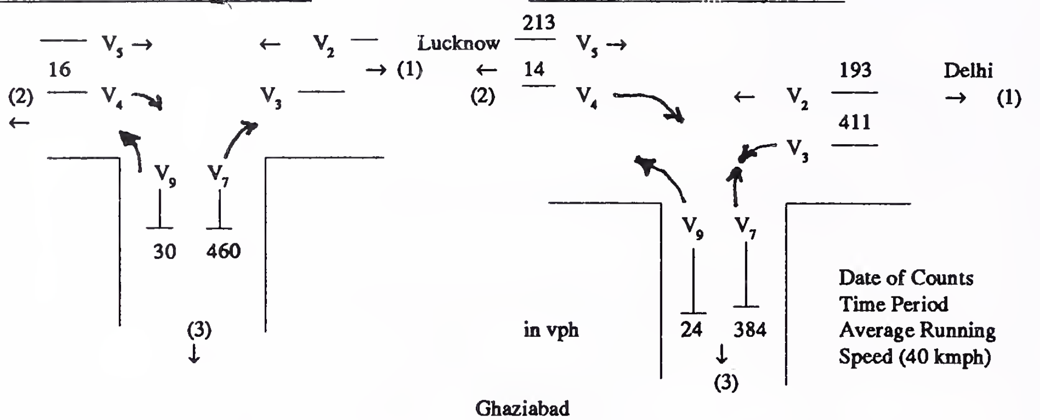
Comments 1. $* 534 = \frac{30}{30/534}$ and so on for other individual movements

$$+ 848 = \frac{30 + 27 + 173}{30/534 + 27/640 + 173/1000}$$

(*) 618 = 848 - (30 + 27 + 173)

2. Above computations indicate that there is no delay (LOS A) from Bulandshahar and Meerut arms of the MDR

Location in Km 20/00	Name Ghaziabad (NH-24/Link Road)
Volume in pcph	Volume in vph — Major Street



VOLUME ADJUSTMENTS

Movement No.	2	3	4	5	7	9
Volume in (vph)	193	411	14	213	384	24
Volume in (pcph)	NA	NA	16	NA	460	30

Step 1 : LT FROM MINOR STREET V_9

Conflicting Flow. Vc	$1/2\ V_3 + V_2 = 206 + 193 = 399$ vph (V_{c9})
Critical Gap. Tc. and Potential Capacity. C_p	Tc = 5.3 sec (Table 2) $C_{p9} = 730$ pcph (Fig. 2)
Actual Capacity. Cm	$C_{m9} = C_{pq} = 730$ pcph

STEP 2 : RT FROM MAJOR STREET V_4

Conflicting Flow. V_c	$1/2\ V_3 + V_2 = 411 + 193 = 604$ vph (V_{c4})
Critical Gap. Tc and Potential Capacity Cp	Tc = 5.4 sec (Table 2) $C_{p4} = 560$ pcph (Fig. 2)
Per cent of Cp Utilized and Impedance Factor (Fig. III-1)	$(V_4/C_{P4}) \times 100 = 2.9\ \%\ P_4 = 0.97$
Actual Capacity : C_m	$Cm_4 = C_{T4} = 560$ pcph

STEP 3 : RT FROM Minor Street V_7

Conflicting flow Vc	$1/2\ V_3 + V_2 + V_5 + V_4 = 206 + 193 + 213 + 14 = 626$ vph (Vc_7)
Critical Gap. Tc and Potential Capacity . Cp	Tc = 6.7 sec (Table 2) $C_{P7} = 380$ pcph (Fig. 2)
Actual Capacity . Cm	$C_{m7} = C_{P7} \times P_4 = 380 \times 0.97 = 369$ pcph

SHARED LANE CAPACITY $C_{SH} = \dfrac{V_7 + V_9}{(V_7/C_{m7}) + (V_9/C_{m9})}$ If lane is shared

Movement No.	pcph	Cm(pcph)		C_{SH} (pcph)		C_R	LOS	
7	460	369	380	369	110	91	D	E
9	30	730		730		700		A
1	16	560				544		A

Comments : Above computations indicate long delays for right turning from Ghaziabad arm (minor road).

Fig. III-6. Worksheet for Analysis of T-intersections