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GEOMETRIC DESIGN STANDARDS FOR URBAN ROADS IN PLAINS

THE INDIAN ROADS CONGRESS

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GEOMETRIC DESIGN STANDARDS FOR URBAN ROADS IN PLAINS

1. INTRODUCTION

1.1. Geometric design deals with the visible elements of a highway. Adoption of proper geometric standards facilitates safe and economical operation of vehicles. Geometric design is influenced by a number of factors among which nature of terrain, type, composition and volume of traffic, operating speed, land-use characteristics and aesthetics are important.

1.2. A draft for this document was initially prepared by the IRC Secretariat. This was considered by the Traffic Engineering Committee (personnel given below) in their meeting held on the 4th and 5th October, 1978 which approved the same subject to certain modifications to be carried out by Dr. N.S. Srinivasan and K. Arunachalam. The draft so modified was approved by the Specifications and Standards Committee in their meeting held on the 24th May, 1983, and later by the Executive Committee and Council in their meetings held on the 21st July, 1983 and 21st August, 1983 respectively.

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2. SCOPE

2.1. These standards are applicable to urban roads in plains. These are also applicable to roads in suburban areas. These however do not cover standards for urban expressways.

2.2. All the main elements of geometric design for urban roads are included in the text. Layout of junctions are not covered as standards for the same are proposed to be brought out separately.

3. CLASSIFICATION OF URBAN ROADS

3.1. For the purpose of geometric design, urban roads other than expressways are classified into four main categories. These are:

(i) Arterial
(ii) Sub-arterial
(iii) Collector Street
(iv) Local Street

This publication deals with standards for all categories of roads except Expressways for which separate standard is proposed to be evolved.

3.2. Definitions

(i) **Arterial :** A general term denoting a street primarily for through traffic, usually on a continuous route.

(ii) **Sub-arterial :** A general term denoting a street primarily for through traffic usually on a continuous route but offering somewhat lower level of traffic mobility than the arterial.

(iii) **Collector Street :** A street for collecting and distributing traffic from and to local streets and also for providing access to arterial streets.

(iv) **Local Street :** A street primarily for access to residence, business or other abutting property.

3.3. Functions

Functions of different categories of urban roads are give below:

(i) **Arterials :** This system of streets, alongwith expresswa where they exist, serves as the principal network f through traffic flows. Significant intra-urban travel such as between central business district and outlying residential areas or between major suburban centres takes place on this system. Arterials should be co-ordinated with existing and proposed expressway systems to provide for distribution and collection of through traffic to and from sub-arterial and collector street systems. Continuity is essential for arterials to ensure efficient movement of through traffic. A properly developed and designated arterial street system would help to identify residential neighbourhoods, industrial sites and commercial areas. These streets may generally be spaced at less than 1.5 km in highly developed central business areas and at 8 km or more in sparsely developed urban fringes. The arterials are generally divided highways with full or partial access. Parking, loading and unloading activities are usually restricted and regulated. Pedestrians are allowed to cross only at intersections.

(ii) **Sub-arterials** : These are functionally similar to arterials but with somewhat lower level of travel mobility. Their spacing may vary from about 0.5 km in the central business district to 3—5 km in the sub-urban fringes.

(iii) **Collector Streets** : The function of collector streets is to collect traffic from local streets and feed it to the arterial and sub-arterial streets or *vice-versa*.

These may be located in residential neighbourhoods, business areas and industrial areas. Normally, full access is allowed on these streets from abutting properties. There are few parking restrictions except during the peak hours.

(iv) **Local Streets** : These are intended primarily to provide access to abutting property and normally do not carry large volumes of traffic. Majority of trips in urban areas either originate from or terminate on these streets. Local streets may be residential, commercial or industrial, depending on the predominant use of the adjoining land. They allow unrestricted parking and pedestrain movements.

3.4. General Considerations

3.4.1. The principal factors to be considered in designating roads into appropriate system are the travel desire lines of people by various modes of transportation, the access needs of adjacent land, network pattern, and existing and proposed land-use.

3.4.2. In designing a road in urban areas, besides the classification of the road, other factors like type of traffic, effect on environment, drainage and maintenance must also be given prime consideration. For example, mixed slow moving traffic requires careful consideration of grades, climbing lanes, curvature etc. Consideration should also be given to see that the road and its structures blend with the environment and produce a pleasing appearance. Noise and fume pollution is a problem in urban areas and the cross-section should provide for remedial measures such as noise barriers, and adequate distance should be kept between busy routes and populated areas. Since idling engines and slow motor vehicles have higher deleterious emissions, arterials should be designed for least stoppages. Design should also take care of drainage, erosion control, space for services and for erecting signs, lighting posts, etc.

4. DESIGN SPEED

4.1. Design speed is related to the function of a road. Keeping in view the type of functions expected of each class of the urban road system, the design speeds given in Table 1 are recommended for adoption.

TABLE 1. DESIGN SPEEDS

Classification	Design Speed (km/hr)
Arterial	80
Sub-arterial	60
Collector street	50
Local street	30

4.2. A lower or higher value compared to that designated in Table 1 may be adopted depending on the presence of physical controls, roadside development and other related factors.

4.3. A lower design speed may be adopted in the central business area or areas with extremely heavy roadside development. On the other hand, in suburban areas, a higher value may be more appropriate.

4.4. For divided highways, running speeds of vehicles are in general higher and, therefore, in such cases a higher value may be adopted.

4.5. It should however be kept in view that sudden change in design speed along any road should be avoided. Change, where necessary, should be made in stages in steps of 10 km/h at a time.

5. SPACE STANDARDS

5.1. The space standards recommended for the various categories of urban roads are given in Table 2.

TABLE 2. RECOMMENDED LAND WIDTHS FOR ROADS IN UUBAN AREAS

Classification	Recommended land width in metres
Arterial	50—60
Sub-arterial	30—40
Collector street	20—30
Local street	10—20

Note : The term "space standard" is often referred to as "right-of-way".

6. CROSS-SECTIONAL ELEMENTS

6.1. The width and layout of urban road cross-sections depend on many factors, the chief amongst them being the classification of road, design speed, and the volume of traffic expected. Other considerations are requirements of parking lanes, bus-bays, loading-unloading bays, occurrence of access points, volume of pedestrians and cyclists, width of drains, location of sewer lines, electricity cables and other public utility services. Plate 1 shows some typical cross-sections. Actual width of each element should be based on traffic volumes and other functional requirements explained in paras 6.2.1 through 6.2.11.

6.2. Road Width and Design Traffic Volumes

6.2.1. The road width should be designed to accommodate the design traffic volume. Past traffic counts and consideration of future development of urban areas must be kept in view while selecting the cross-section of road. Estimation of future traffic volumes may be based on a simple projection of current volumes extrapolated from past trends, or on the basis of results of transportation study which allows for change in land-use and accounts for socio-economic factors. The road should be designed to accommodate the traffic volumes computed for the end of design life. A design period of 15-20 years should be adopted for arterials and sub-arterials and 10-15 years for local and collector streets. A higher design period should be taken for small towns and a lower design period for large cities. For high volume streets and busy intersections, peak hour volumes should be used to determine the widths. For rough estimate, the peak hour flows may be taken as 10-12 per cent of the daily flow.

6.2.2. Traffic in urban areas in the country is of mixed nature. The width requirement should be assessed on the basis of equivalent passenger car units (PCU) using the tentative equivalency factors shown in Table 3.

TABLE 3. PASSENGER CAR EQUIVALENCY FACTORS

S. No.	Vehicle Type	Equivalency Factor
1.	Passenger car, tempo, auto-rickshaw, Jeep, van or agricultural tractor	1.0
2.	Truck, bus or agricultural tractor-trailer	3.0
3.	Motor-cycle, scooter and cycle	0.5
4.	Cycle-rickshaw	1.5
5.	Horse-drawn vehicle	4.0
6.	Bullock-cart	8.0*
7.	Hand-cart	6.0

*For smaller bullock-cart, a value of 6 will be appropriate.

As the influence of different types of vehicle on the capacity of through urban roads at different situations such as through sections, roundabouts and intersections is different, the equivalency factors for different situations are different. The equivalency factors given above are applicable only to through sections of urban roads between junctions.

6.2.3. The design of main traffic routes in built-up areas should be based on peak hour demands and not as in rural areas on the average daily traffic. On two-way undivided carriageway, the capacity is relatively independent of distribution by direction, and design is based on two-way total flows. On dual or divided carriageway, capacity is dependent on distribution by direction and design should therefore be based on peak hour flow in the busier direction of travel. Tentative practical capacities for both uni-direction and two-direction flows of urban roads between junctions are given in Table 4.

TABLE 4. TENTATIVE CAPACITIES OF URBAN ROADS BETWEEN INTERSECTIONS

No. of traffic lanes and widths	Traffic flow	Capacity in PCUs per hour for various traffic conditions		
		Roads with no frontage access, no standing vehicles, very little cross traffic	Roads with frontage access but no standing vehicle and high capacity intersections	Roads with free frontage access, parked vehicles and heavy cross traffic
2-lane (7-7.5m)	One way	2400	1500	1200
	Two way	1500	1200	750
3-lane (10.5m)	One way	3600	2500	2000
4-lane (14 m)	One way	4800	3000	2400
	Two way	4000	2500	2000
6-lane (21 m)	One way*	3600	2500	2200
	Two way	6000	4200	3600

*For three lanes in predominant direction of flow.

6.2.4. **Carriageway width** : Recommendedc arriageway widths are shown in Table 5.

TABLE 5. RECOMMENDED CARRIAGEWAY WIDTHS

Description	Width (metres)
Single lane without kerbs	3.50
2-lane without kerbs	7.00
2-lane with kerbs	7.50
3-lane with or without kerbs	10.5/11.0
4-lane with or without kerbs	14.0
6-lane with or without kerbs	21.0

Notes : 1. For access roads to residential areas, a lower lane width of 3 m is permissible.

2. Minimum width of a kerbed urban road is 5.5 m including allowance for a stalled vehicle.

6.2.5. **Footpath (Sidewalk)** : The minimum width of footpath should be 1.5 metres. They should have well maintained surface with crossfall neither so flat as to be difficult to drain nor so steep as to be dangerous to walk upon. The crossfall within the range of 2.5 to 3 per cent should meet this requirement. Those parts of the footpath immediately adjoining buildings, fences, trees and other obstructions, which will not be available for free movement of pedestrians should be disregarded while calculating widths required. Table 6 gives the capacity guidelines for design of footpaths.

TABLE 6. CAPACITY OF FOOTPATHS

Number of person per hour		Required width of footpath (metre)
All in one direction	In both directions	
1200	800	1.5
2400	1600	2.0
3600	2400	2.5
4800	3200	3.0
6000	4000	4.0

The width should be increased by 1 metre in business and shopping areas to allow for dead width. Footpaths adjoining shopping frontages should be atleast 3.5 m and a minimum of 4.5 m is desirable adjoining longer shopping frontages. At points of possible congestion such as bus stop or entrance of large shops and public buildings, footpaths may be wider. Where space is available, provision of verge between footpath and carriageway to increase safety of pedestrians is desirable. When deciding the width of footpaths and verges, the width required to accommodate underground services clear of carriageway should also be taken into account. When on slopes or in the case of ramps, the capacity should be suitably reduced.

6.2.6. **Cycle Track** : The minimum width of cycle track should be 2 metres. Each additional lane where required should be 1 m. Separate cycle tracks should be provided when the peak hour cycle traffic is 400 or more on routes with a motor vehicle traffic of 100-200 vehicles per hour. When the number of motor vehicle using the route is more than 200 per hour, separate cycle tracks are justified even if cycle traffic is only 100 per hour. As a general rule, the capacity of cycle tracks may be taken as given in Table 7.

TABLE 7. CAPACITY OF CYCLE TRACKS

Width of cycle track		Capacity in number of cycles/hour	
		One-way traffic	Two-way traffic
Two lanes	(3 m)	250 to 600	50 to 250
Three lanes	(4 m)	over 600	250 to 600
Four lanes	(5 m)	——	over 600

6.2.7. **Medians** : Urban highways of six lanes or more should, as a general rule, be provided with median. For four-lane roads, however, the provision of median should be judicious taking into account such considerations as safety, directional distribution of traffic, the proportion of slow-moving traffic, roadside development and quality of service, etc. As far as possible, medians should be avoided where there are significant tidal flows of traffic, or where the individual carriageways are inadequate for catering to peak-hour traffic volumes, or where there is intense roadside developments without frontage roads.

Width of median is dictated by a variety of conditions. Widths will depend on the available right-of-way, terrain, turn lanes, drainage and other determinants. Wide medians are preferred where space and cost considerations permit. Minimum widths of median at intersections to accomplish various purposes should be as follows : (i) Pedestrian refuge, 1.2 m; (ii) Median lane for protection of vehicle making right turn, 4.0 m but 7.5 m is recommended; (iii) 9 to 12 metre is required to protect vehicles crossing at grade. Even greater widths are required for U-turns. Absolute minimum width of median in urban areas is 1.2 m; a desirable minimum is 5 m.

As far as possible, the median should be of uniform width in a particular section. However, where changes are unavoidable, a transition of 1 in 15 to 1 in 20 must be provided.

6.2.8. **Verge** : Verges are required between carriageway and property line not only to accommodate lighting columns, traffic signs, underground services etc., but also to provide appropriate clearance to ensure proper vehicle placement and development of full carriageway capacity. Where road width is restricted, full width between carriageway and property line should be paved and used for pedestrian sidewalk/cycle track. Where possible, a minimum verge of 1 m width should be kept. They should be suitably levelled, trimmed and provided with a crossfall of 5 per cent if turfed and 3 per cent if cobbled or surface dressed. This should be increased if poles, kerb-height, or excessive crossfall discourage parking close to the kerb and also where either parked vehicles frequently overlap on to the adjacent traffic lane or the parking lane is likely to be used as a peak hour traffic lane.

6.2.9. **Parking lanes :** Parking lanes may be provided on all sub-arterials and collector streets in business and shopping areas. Parallel kerb parking should be preferred. Parking lane width for parallel parking should be 3 m which may be reduced to 2.5 m where available space is limited. Where additional parking capacity is desired and sufficient carriageway width is available, angle parking may be adopted.

6.2.10. **Busbays :** Busbays should not be located too close to intersections. It is desirable that they are located 75 m from the intersection on either side preferably on the farther side of the intersection.

Busbays should be provided preferably by recessing the kerb to avoid conflict with moving traffic. The length of the recess should be 15 m for single bus stop with increase of 15 m for each extra bus for multiple bus stops. The taper should be desirably 1 : 8 but not less than 1 : 6. The depth of the recess should be 4.5 m for single bus stop and 7 m for multiple bus stop. Suitable arrangement should be made for drainage of surface water from busbays. Sufficient footpath should be ensured behind the busbays.

6.2.11. **Lay-byes :** To enable drivers to stop clear of carriageway, lay-byes should be provided at intervals along long straight routes. They should always be provided near guide maps and other public conveniences to enable drivers to stop clear of carriageway. They should normally be 3 m wide and atleast 30 m long with 15 m end tapers on both sides. Suitable arrangements should be made for drainage of surface water from lay-byes.

7. KERB

7.1. It is desirable that roads in urban areas are provided with kerbs.

7.2. Kerbs may be barrier type, semi-barrier type or mountable type. Appropriate situations for use of each type is indicated below :

(a) Barrier type [Fig. 1(a)]	:	Built-up areas adjacent to footpaths with considerable pedestrain traffic.
(b) Semi-barrier type [Fig. 1(b)]	:	On the periphery of the roadway where pedestrain traffic is light and a barrier type could tend to reduce traffic capacity.
(c) Mountable type [Fig. 1(c)]	:	Within the roadway at channelization schemes, medians, outer separators and raised medians on bridges.

7.3. Each figure shows two varieties of each type of kerb with gutter and without gutter. Kerbs with gutter should always be used at drainage edges of pavements.

8. CAMBER

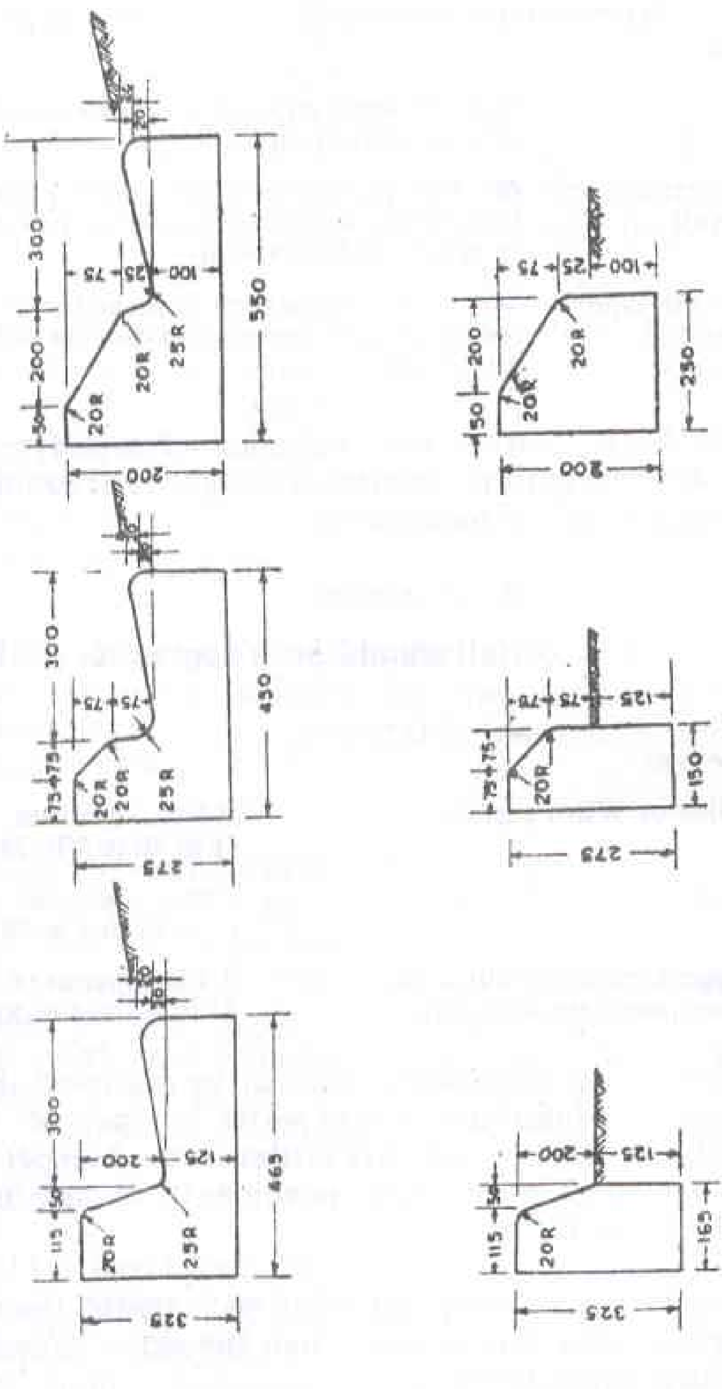
8.1. Camber or crossfall should be adopted as follows for straight sections :

Surface type	*Camber*
(i) Gravelled or WBM surface	2.5 to 3 per cent (1 in 40 to 1 in 33)
(ii) Thin bituminous surfacing	2 to 2.5 per cent (1 in 50 to 1 in 40)
(iii) High type bituminous surfacing or cement concrete surfacing.	1.7 to 2 per cent (1 in 60 to 1 in 50)

8.2. Higher values of camber should be adopted in areas with high intensity of rainfall and where water is expected to pond in local depressions due to unequal settlement. Steeper camber should also be provided on kerbed pavements to minimise the spread of surface water flows.

8.3. For shoulders along unkerbed pavements, the crossfall should be of least 0.5 per cent steeper than the slope of pavement subject to minimum given below :

WBM surface	3 per cent
Gravel surface	4 per cent
Earth surface	5 per cent



(a) Barrier type

(b) Semi-barrier type

(c) Mountable type

All dimensions in millimetres

Fig. 1. Typical kerb sections

8.4. For paved footpaths, crossfall of 3-4 per cent should be adopted.

8.5. For verges and unpaved areas, the crossfall should be 4-6 per cent.

8.6. Undivided carriageways should have a crown in the middle and slope towards the edges.

8.7. Divided roads may have a single crowned section or separate crowned sections for each carriageway depending on requirements of drainage and access to abutting property.

9. SIGHT DISTANCE

9.1. Stopping sight distance should be provided at all points on the road. Stopping sight distance is the total distance travelled by the driver from the time a danger is comprehended by him to the actual stop, i.e. the distance travelled during perception and brake reaction time plus the braking distance. For the purpose of measuring the stopping sight distance, the height of eye should be assumed as 1.2 m and height of object as 0.15 m.

The design values of sight distance are shown in *Table 8*.

TABLE 8. SAFE STOPPING SIGHT DISTANCE FOR VARIOUS SPEEDS*

Speed (km/h)	Safe stopping sight distance (metre)
30	30
50	60
60	80
80	120

*For other design speeds, see IRC : 66-1976

9.2. On undivided roads, intermediate sight distance which is equal to twice the stopping distance should be provided where vehicles are permitted to cross the centre line.

9.3. Headlight Sight Distance

On valley curves, the design must ensure that the roadway ahead is illuminated during night travel by vehicle headlights for a sufficient length which enables the vehicle to brake to a stop, if necessary. This is known as the headlight sight distance and is equal to the safe stopping distance. From safety considerations, valley curves should be designed to provide for this visibility.

For designing valley curves, the following criteria should be followed to ensure the headlight sight distance:

(i) height of headlight above the road surface is 0.75 m;

(ii) the useful beam of headlight is one degree upwards from the grade of the road; and

(iii) the height of object is nil.

10. HORIZONTAL ALIGNMENT

10.1. In general, horizontal curves should consist of a circular portion flanked by spiral transitions at both ends. Design speed, superelevation and coefficient of side friction affect the design of circular curves. Length of transition curves is determined on the basis of rate of change of centrifugal acceleration and superelevation.

10.2. Superelevation

10.2.1. **Design values**: Superelevation required on horizontal curves should be calculated from the following formula. This assumes the centrifugal force corresponding to three-fourth the design speed is balanced by superelevation and rest counteracted by side friction:

$$e = \frac{V^2}{225R}$$

where

e = superelevation in metre per metre

V = speed in km/h, and

R = radius in metres

Superelevation obtained from the above expression should be limited to 7 per cent. However, on urban sections with frequent intersections, it will be desirable to limit the superelevation to 4 per cent for convenience in construction and for facilitating easy and safe turning movement of vehicles.

Fig. 2 indicates the superelevations for various design speeds on this basis.

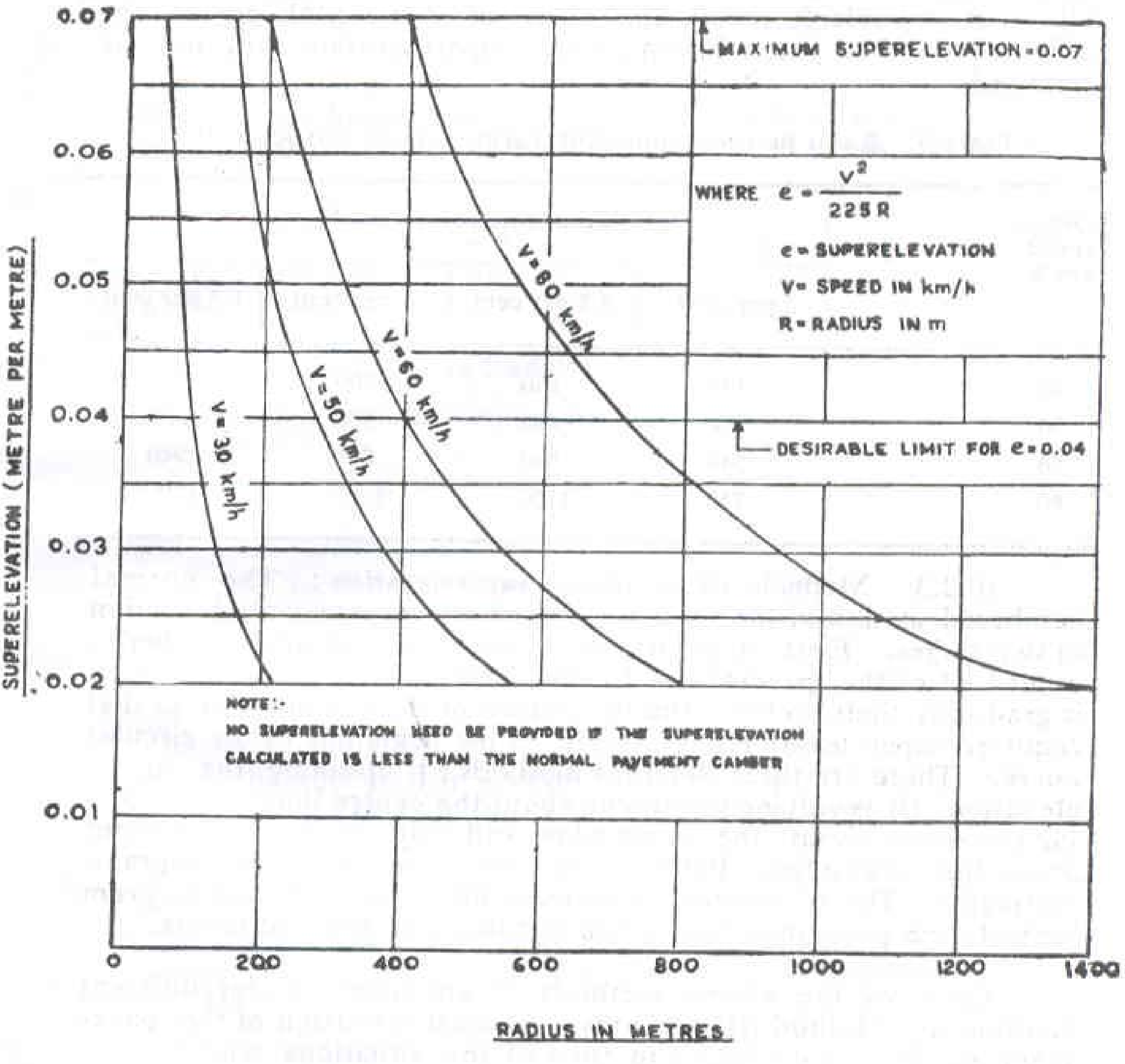


Fig. 2. Superelevation for various design speeds

10.2.2. **Radii beyond which no superelevation is required :** When the value of the superelevation obtained vide para 10.2.1 is less than the road camber, the normal cambered section should be continued on the curved portion without providing any superelevation. Table 9 shows the radii of horizontal curves for different camber rates beyond which superelevation will not be required.

TABLE 9. RADII BEYOND WHICH SUPERELEVATION IS NOT REQUIRED

Design speed km/h	Radius (metre) for camber of			
	3 per cent	2.5 per cent	2 per cent	1.7 per cent
30	130	160	200	240
50	370	450	550	650
60	540	640	800	940
80	950	1100	1400	1700

10.2.3. **Methods of attaining superelevation** : The normal cambered section of the road is changed into superelevated section in two stages. First stage is the removal of adverse camber in outer half of the pavement. In the second stage, superelevation is gradually built-up over the full width of the carriageway so that required superelevation is available at the beginning of the circular curve. There are three different methods for attaining the superelevation: (i) revolving pavement about the centre line; (ii) revolving pavement about the inner edge; and (iii) revolving pavement about the outer edge. Plate 2 illustrates these methods diagrammatically. The small cross-sections at the bottom of each diagram indicate the pavement cross-slope condition at different points.

Each of the above methods is applicable under different conditions. Method (i) which involves least distortion of the pavement will be found suitable in most of the situations where there are no physical controls, and may be adopted in th the normal course. Method (ii) is preferable where the lower edge profile is a major control, e.g. on account of drainage. Where overall appearance is the criterion, method (iii) is preferable since the outer edge profile which is most noticeable to drivers is not distorted.

The superelevation should be attained gradually over the full length of the transition curve so that the design superelevation is available at the starting point of the circular portion. Sketches in Plate 2 have been drawn on this basis. In cases where transition curve cannot for some reason be provided, two-third superelevation may be attained on the straight section before start of the circular curve and the balance one-third on the curve.

In developing the required superelevation, it should be ensured that the longitudinal slope of the pavement edge compared to the centerline (i.e. the rate of change of superelevation) is not steeper than 1 in 150.

When cross-drainage structures fall on a horizontal curve, their deck should be superelevated in the same manner as described above.

10.3. Minium Curve Radius

Minimum radius of curve can be determined from the equation :

$$R = \frac{V^2}{127\,(e+f)}$$

where

V = vehicle speed in km/h

e = superelevation ration in metre per metre

f = coefficient of side friction between vehicle tyres and pavement (taken as 0.15)

R = radius in metres

Based on this equation, minimum radii of horizontal curves for the different design speeds with maximum superelevation limited to 4 per cent and 7 per cent are given in Table 10.

TABLE 10. MINIMUM RADII OF HORIZONTAL CURVES

Design speed km/h	Minimum radius (metre) when superelevation is limited to	
	7 per cent	4 per cent
30	30	40
50	90	105
60	130	150
80	230	265

10.4. Set-back Distance at Horizontal Curves

Physical obstructions on the inside of horizontal curves often restrict sight distance. Sight areas on horizontal curves should be such as to provide driver with sight distance equal to the design stopping distance on curve. Figure 3 indicates the minimum width of set back from obstructions to sight measured from centre line of innermost lane. These values are only applicable when the length of arc of the curve is greater than the design stopping distance. For shorter lengths of curves, width of sight area should be checked by trial and error by assuming various positions of object and drivers on straight portions adjoining the curve.

10.5. Transition Curves

10.5.1. Transition curves are necessary for a vehicle to have smooth entry from a straight section into a circular curve. The transition curves also improve aesthetic appearance of the road besides permitting gradual application of the superelevation and extra widening of carriageway needed at the horizontal curves. Spiral curve should be used for this purpose.

10.5.2. Minimum length of the transition curve should be determined from the following two considerations and the larger of the two values adopted for design:

(i) The rate of change of centrifugal acceleration should not cause discomfort to drivers. From this consideration, the length of transition curve is given by :

$$L_s = \frac{0.0215\, V^3}{CR}$$

where

L_s = length of transition in metres

V = speed in km/h

R = radius of circular curve in metres

$C = \frac{80}{75+V}$ (subject to a maximum of 0.8 and minimum of 0.5)

(ii) The rate of change of superelevation (i.e. the longitudinal grade developod at the pavement edge compared to through grade along the centre line) should be such as not to cause discomfort to travellers or to make the road appear unsightly. This rate of change should not be steeper than 1 in 150. The formula for minimum length of transition on this basis with superelevation limited to 7 per cent works out to :

$$L_s = \frac{2.7\, V^2}{R}$$

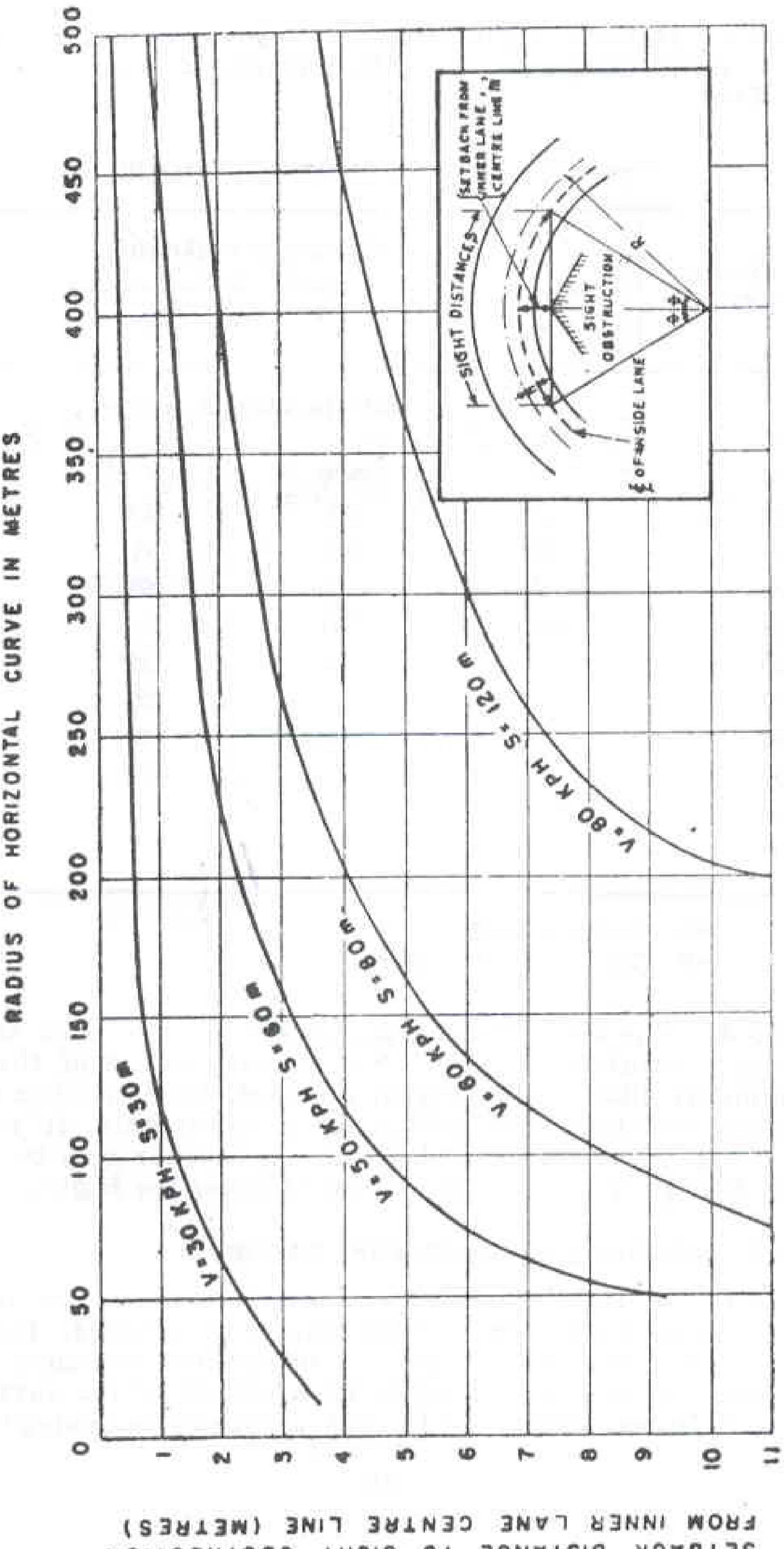


Fig. 3. Minimum set-back distance required at horizontal curves on two lane urban roads for safe stopping sight distance

10.5.3. Having regard to the above considerations, the minimum transition lengths for different speeds and curve radii are given in Table 11.

TABLE 11. MINIMUM TRANSITION LENGTHS

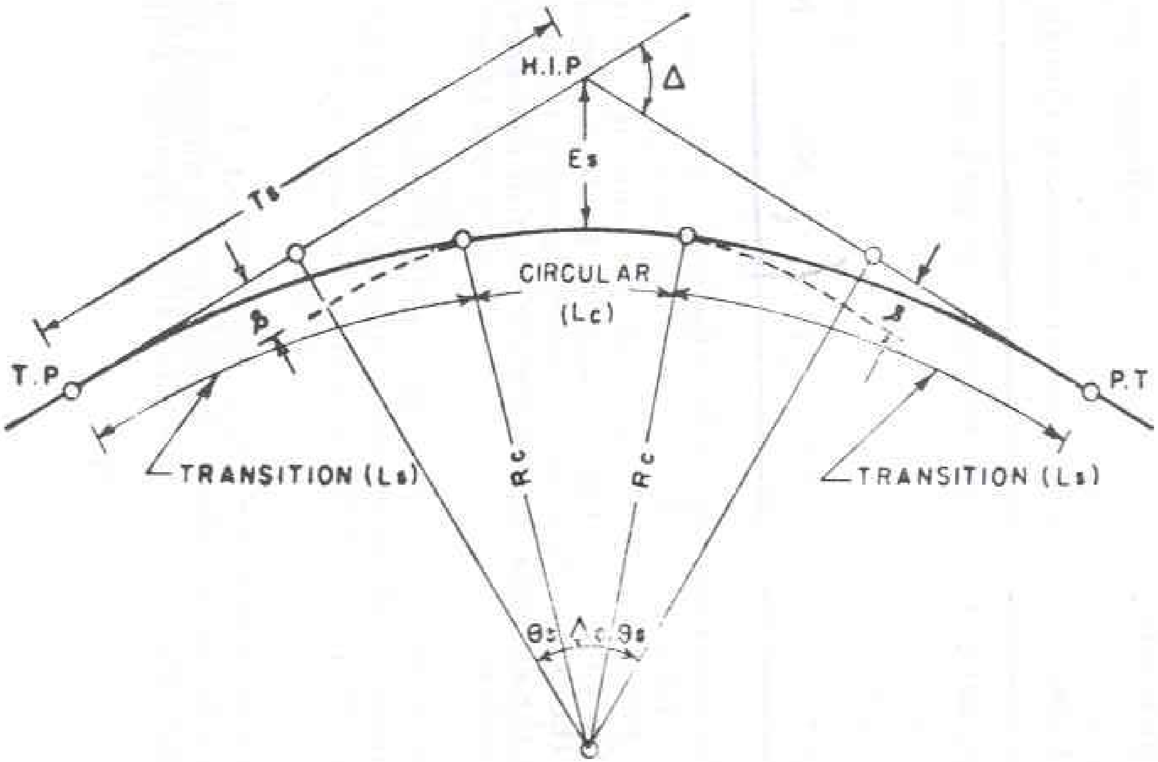
Curve radius R (metre)	Design speed (km/h)			
	30	50	60	80
	Transition length—metre			
30	80			
50	50	NA		
100	25	70	NA	
150	20	45	65	
200	15	35	50	NA
250	NR	30	40	85
300		25	35	75
400		20	25	55
500		NR	20	45
600			20	35
800			NR	30
1000				30

NA—Not applicable
NR—Transition not required

10.5.4. The elements of a combined circular and transition curves are illustrated in Fig. 4. For deriving values of the individual elements like shift, tangent distance, apex distance ete. and working out coordinates to lay the curves in the field, it is convenient to use curve tables. For this, reference may be made to IRC: 38 "Design Tables for Horizontal Curves for Highways".

10.6. Widening of Carriageway on Curves

10.6.1. At sharp horizontal curves, it is necessary to widen the carriageway to provide for safe passage of vehicles. The widening required has two components: (i) mechanical widening to compensate the extra width occupied by a vehicle on the curve due to tracking of the rear wheels, and (ii) psychological widening to permit



TANGENT POINT.	T.P, P.T.
HORIZONTAL INTERSECTION POINT.	H.I.P.
TOTAL DEVIATION ANGLE. . . .	Δ
DEVIATION AND CENTRAL ANGLE OF CIRCULAR ARC. . . .	Δ_c
DEVIATION ANGLE OF TRANSITION CURVE.	θ_s
RADIUS OF CIRCULAR CURVE	R_c
SHIFT.	S
TANGENT DISTANCE.	T_s
APEX DISTANCE	E_s
LENGTH OF TRANSITION.	L_s
LENGTH OF CIRCULAR CURVE. .	L_c

Fig. 4. Elements of a combined circular and transition curve

easy crossing of vehicles since vehicles in a lane tend to wander more on a curve than on a straight reach.

10.6.2. On two-lane or wider roads, it is necessary that both the above components should be fully catered for so that the lateral clearance between vehicles on curves is maintained equal to the clearance available on straights. Position of single-lane roads however is somewhat different, since during crossing manoeuvres outer wheels of vehicles have in any case to use the shoulders whether on the straight or on the curve. It is, therefore sufficient on single lane roads if only the mechanical component of widening is taken into account.

10.6.3. Based on the above considerations, the extra width of carriageway to be provided at horizontal curves on single and two-lane roads is given in Table 12. For multi-lane roads, the pavement widening may be calculated by adding half the widening for two-lane roads to each lane.

Table 12. Extra Width of Pavement at Horizontal Curves

Radius of curve (m)	Upto 20	21 to 40	41 to 60	61 to 100	101 to 300	above 300
Extra width (m)						
Two-lane	1.5	1.5	1.2	0.9	0.6	Nil
Single-lane	0.9	0.6	0.6	Nil	Nil	Nil

10.6.4. The widening should be effected by increasing the width at an approximately uniform rate along the transition curve. The extra width should be continued over the full length of the circular curve. On curves having no transition, widening should be achieved in the same way as the superelevation i.e. two-third being attained on the straight section before start of the curve and one-third on the curve.

10.6.5. The widening should be applied equally on both sides of the carriageway. However the widening should be provided only on the inside when the curve is plain circular and has no transition.

10.6.6. The extra widening may be attained by means of offsets radial to the centre line. It should be ensured that the pavement edge lines are smooth and there is no apparent kink.

11. VERTICAL ALIGNMENT

11.1. General

Vertical alignment in urban areas is governed by need to match building line and entrance line levels and levels of intersections and median openings.

11.2. Gradient

Most urban roads carry mixed traffic including slow moving vehicles like bicycles and animal/hand carts. Besides this, urban roads generally have intersections at frequent intervals. In view of this, as a general rule, a gradient of 4 per cent should be considered the maximum for urban roads. On roads carrying predominantly slow moving traffic, however, the gradient should desirably not exceed 2 per cent. At intersections, the road should be as near level as possible.

As the urban roads are generally kerbed, it would be desirable to ensure a minimum gradient as indicated in Table 13 for facilitating longitudinal drainage.

TABLE 13. RECOMMENDED MINIMUM GRADIENTS

Design Element	Gradient	
	Desirable minimum (per cent)	Absolute minimum (per cent)
Kerbed Pavements	0.5	0.3
Side ditches (lined)	0.5	0.2

The desirable maximum gradients for pedestrain ramps and cycle tracks are as follows :

Pedestrain ramps	10 per cent
Cycle tracks	3 per cent

11.3. Vertical Curves

Vertical curves should be provided at all grade changes exceeding those indicated in Table 14. For satisfactory appearance, the minimum length should be as shown in Table 14 between changing grade lines. The minimum lengths of vertical curves and maximum grade change without a vertical curve are shown in Table 14.

TABLE 14. MINIMUM LENGTH OF VERTICAL CURVES

Design speed (km/h)	Maximum grade change (per cent) not requiring a vertical curve	Minimum length of vertical curve (m)
30	1.5	15
50	1.0	30
60	0.8	40
80	0.6	50

11.4. Summit Curves

Summit curves in urban areas should be designed for safe stopping sight distance and they should be coordinated with horizontal curvature. Broken-back profiles should be avoided and wherever possible, approaches to bridges less than 30 m width should be designed to fit a single vertical curve.

Length of the summit curve should be calculated on the basis of the following formulae :

(i) **When the length of the curve exceeds the required sight distance i,e. L is greater than S**

$$L = \frac{NS^2}{4.4}$$

Where N = deviation angle, i.e. the algebraic difference between two grades

L = length of vertical curve in metres

S = sight distance in metres.

(ii) **When the length of the curve is less than the required sight distance i.e. L is less than S**

$$L = 2S - \frac{4.4}{N}$$

The minimum length of summit curves for stopping sight distance and various deviation angles have been calculated and given in Fig. 5. Summit curves shall be square parabolas ($y = ax^2$) and minimum length should not be less than that given in Table 14.

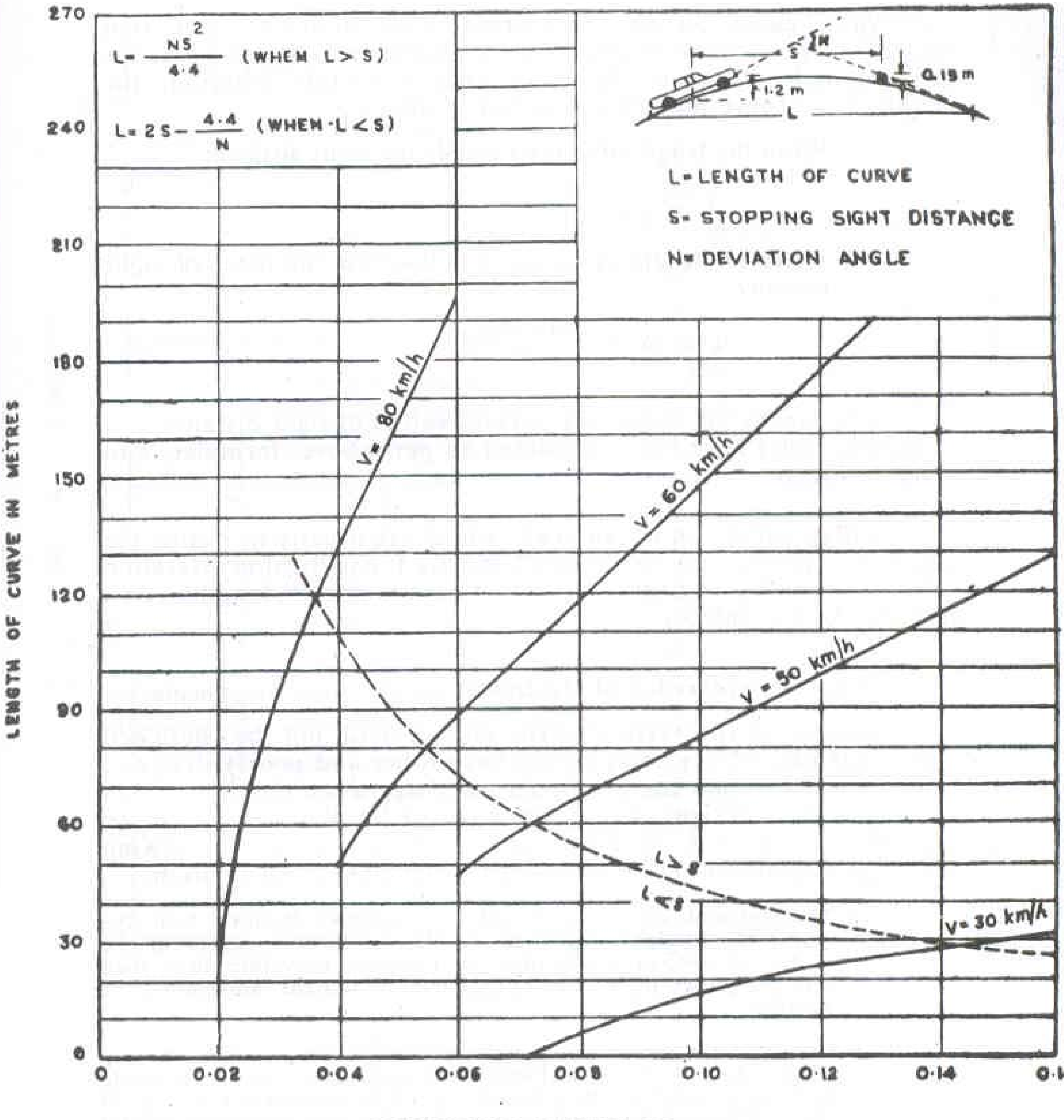


Fig. 5. Length of summit curve for stopping sight distance

11.5. Valley Curves

Valley curves on unlighted urban roads should be such that for night travel the headlight beam distance is the same as the stopping sight distance. In accordrnce with this criterion, the length of the curve may be calculated as under :

(i) **When the length of curve exceeds the sight distance**

$$L = \frac{NS^2}{1.50 + 0.035S}$$

(ii) **When the length of the curve is less than the requiied sight distance**

$$L = 2S - \frac{1.50 + 0.035S}{N}$$

The length of curves for various values of sight distance and deviation angles have been calculated as per above formulae and given in Fig. 6.

Valley curves on urban roads which are usually lit during the hours of darkness may be designed merely for vertical acceleration of 0.5 g for riding comfort and in that case minimum lengths given in Table 14 will suffice.

11.6. Co-ordination of Horizontal and Vertical Alignments

Horizontal and vertical alignments should not be designed independently. They complement each other and poorly designed combination can mar the good points and aggravate the deficiencies of each. The design should be visualised in the perspective to achieve a flowing and pleasing view from the road. Following broad principles should be followed in alignment co-ordination:

(i) The degree of curvature should be in proper balance with the gradients. Straight alignment or flat horizontal curves at the expense of steep or long grades, or excessive curvature in a road with flat grades, do not constitute balanced designs and should be avoided.

(ii) Vertical curve superimposed upon horizontal curve gives a pleasing effect. As such the vertical and horizontal curves should coincide as far as possible and their lengths should be more or less equal. If this is difficult to achieve for any reason, the horizontal curve should be somewhat longer than the vertical curve.

(iii) Sharp horizontal curves should be avoided at or near the apex of pronounced summit/sag vertical curves from safety considerations.

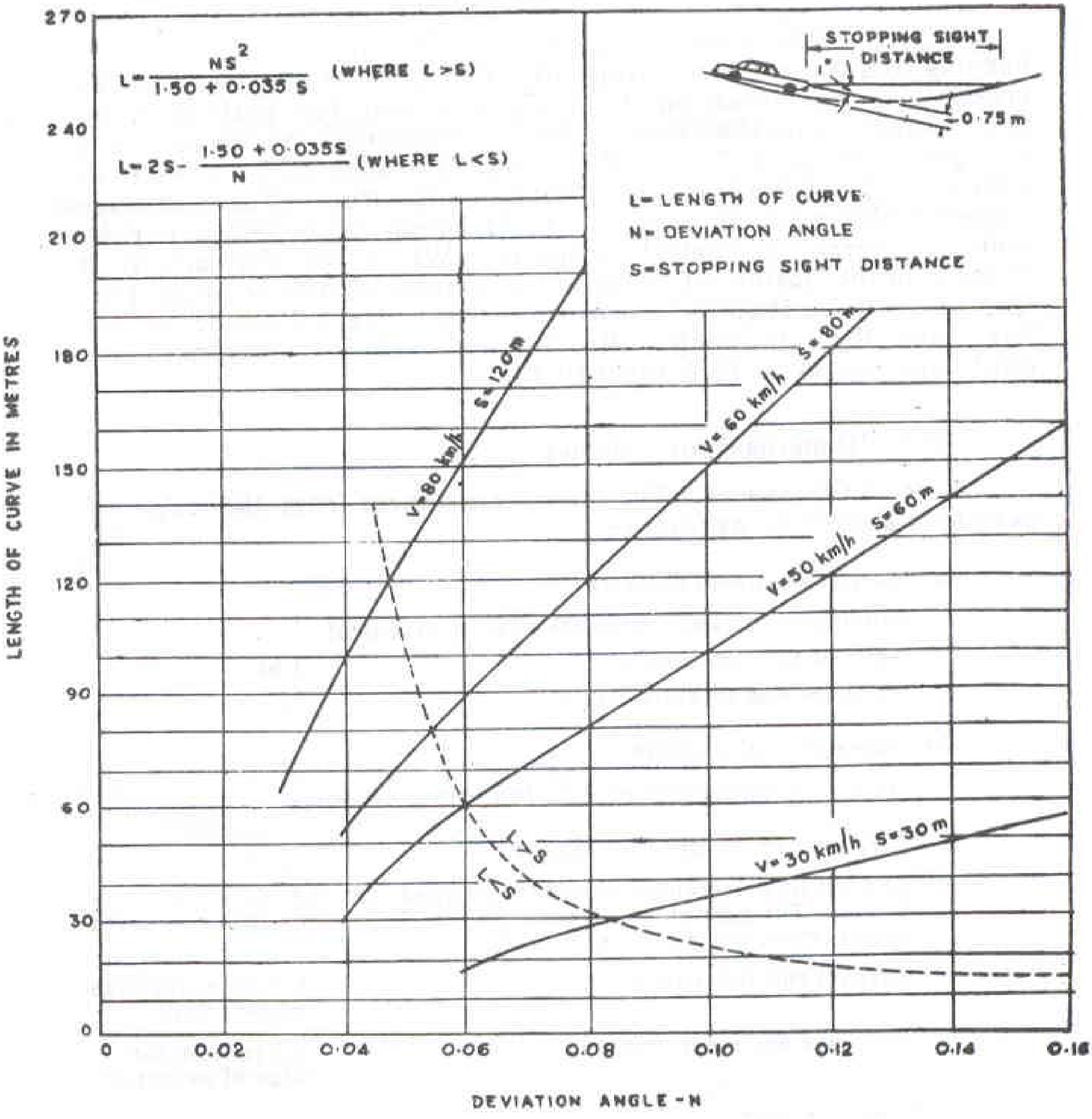


Fig. 6. Length of valley curve

12. CLEARANCES

12.1. Clearances are required to be provided for overhanging loads and the tilting of vehicle towards obstruction by crossfall or superelevation of carriageway and for kerb shyness. Standards for lateral clearances for underpasses on urban roads are given in para 7 of IRC : 54-1974 "Lateral and Vertical Clearances at Underpasses for Vehicular Traffic". The same are recommended between edge of carriageway and obstruction on footpath, verge or central reserve. Where an obstruction is located on the inside of a bend, a greater clearance than that specified may be required to ensure that the sight distance is not less than the minimum. Broad standards for clearances are reproduced in paras 12.2. through 12.5.

12.2. Underpass for Vehicles

Lateral Clearances : The lateral clearances from the edge of pavement should be as follows :

(a) *Pavement without footpath*

Minimum clearances from the edge of pavement

Arterial and sub-arterial	...	1 m
Collector and local streets	...	0.5 m

(b) *Pavement with footpath*

No extra clearance beyond the footpath is necessary.

(c) *Clearance on divided carriageway*

The left side clearances should be followed on the same lines as above. The right side clearance to the face of any structure in the central median shall be as follows :

Arterial and sub-arterial	...	1 m from the edge of pavement
Collector and local streets	...	0.5 m from the edge of pavement

Vertical Clearance

Minimum vertical clearance on urban roads should be 5.5 m.

12.3. Pedestrian Subway

The minimum width of pedestrian subway is 2.5 metres. The minimum vertical clearance over such subway is 2.5 m.

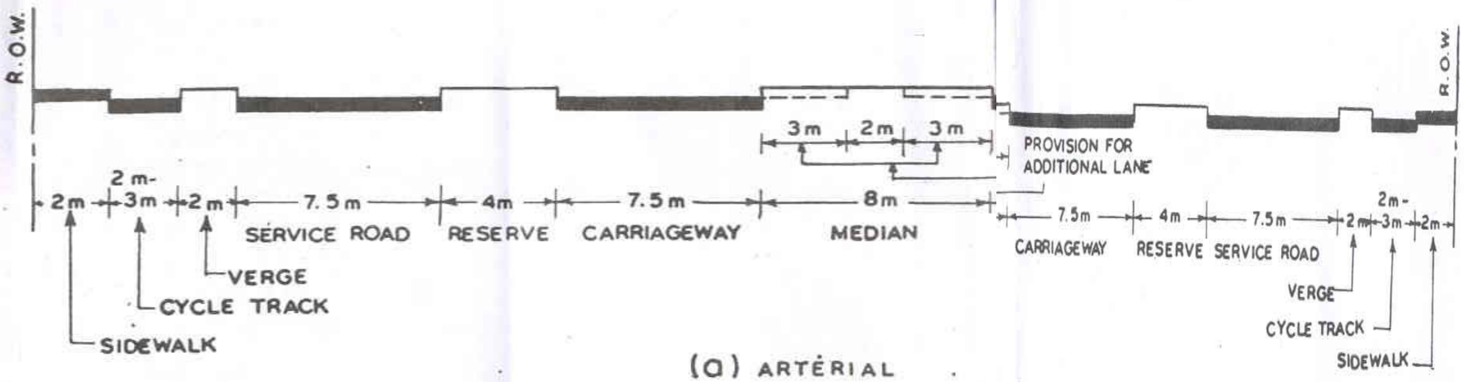
12.4. Cycle Subway

The minimum width of underpass for cycles is 2.5 m. The minimum vertical clearance for cycle tracks is 2.5 m.

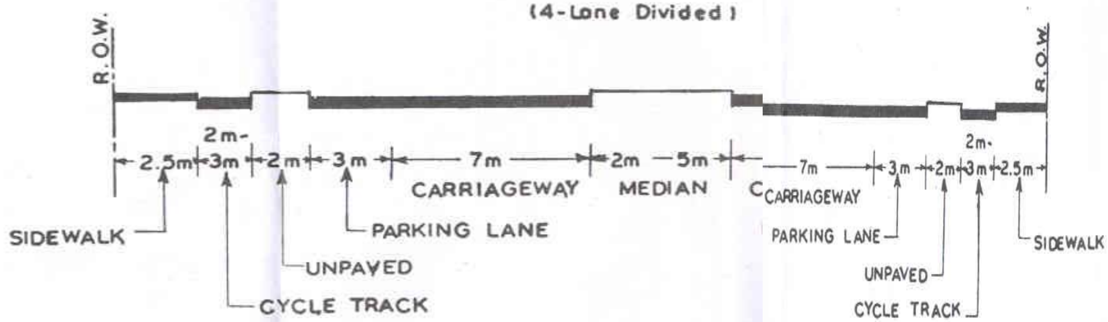
12.5. Combined Cycle and Pedestrian Subway

The width of pedestrian-cum-cycle subway should be 5 m minimum for one-way traffic and 6.5 m for two-way traffic. The minimum height should be 2.5 m.

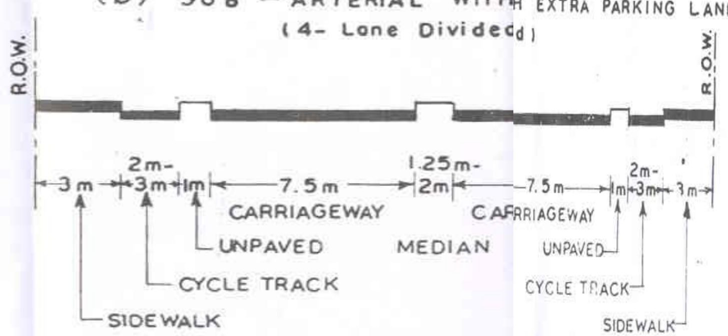
Plate 1



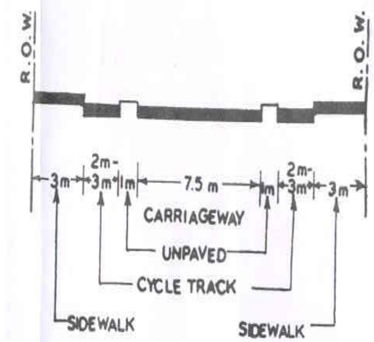
(a) ARTERIAL
(4-Lane Divided)



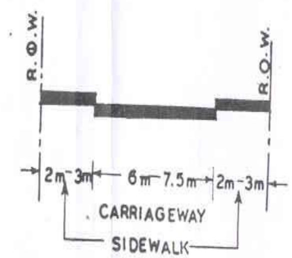
(b) SUB - ARTERIAL WITH EXTRA PARKING LANE
(4- Lane Divided)



(C) COLLECTOR STREET
(4-Lane Divided)



(d) COLLECTOR STREET
(2- Lane)

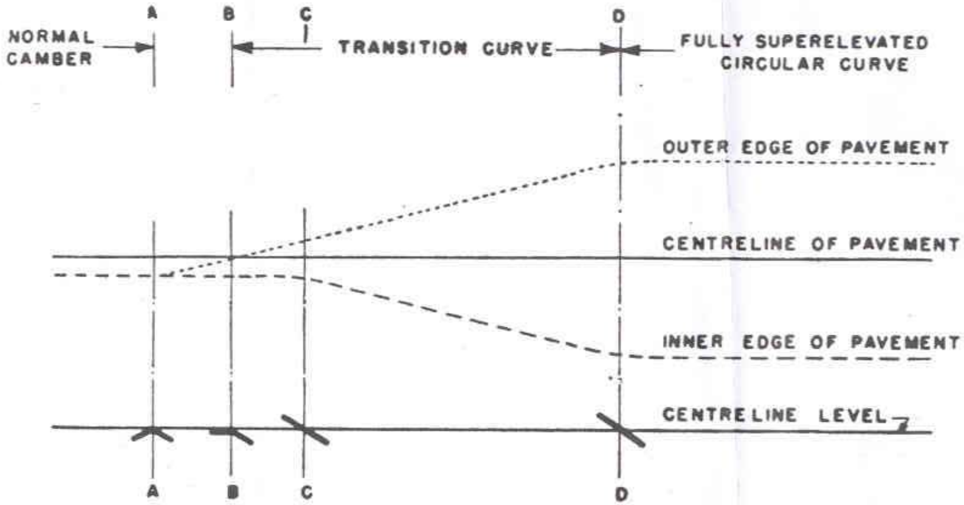


(e) LOCAL STREET WITH SIDEWALK

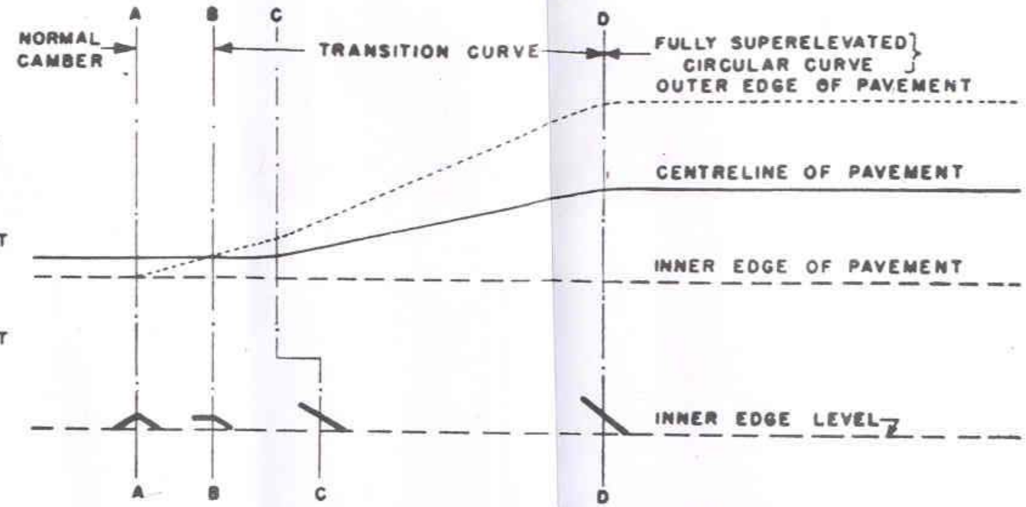
Notes:
1. These are only typical cross-sections. Right-of-way limits and individual elements of cross-section may vary according to needs and recommendations in para 6 of the Standard
2. Position of roadside trees and lighting poles etc. is not shown. These, can be suitably fitted according to situations
3. For more details, reference may be made to IRC : 69-1977.

TYPICAL CROSS-SECTIONS OF URBAN ROADS

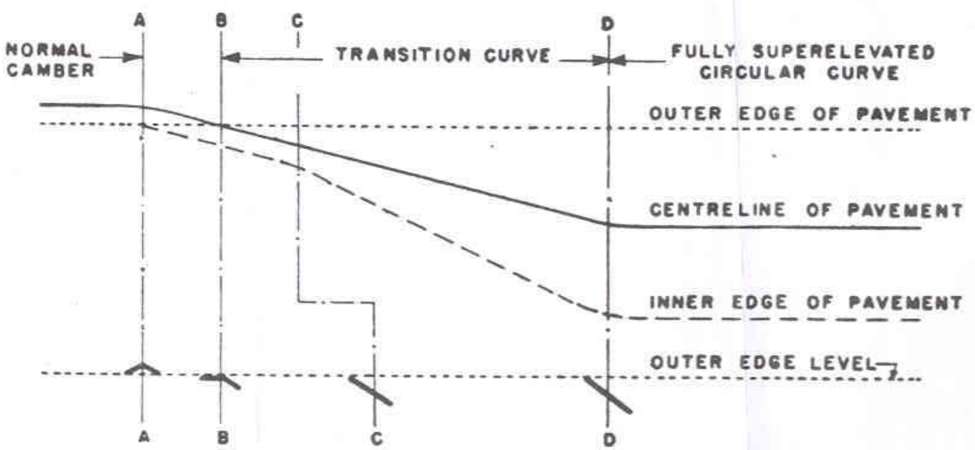
Plate 2



(a) PAVEMENT REVOLVED ABOUT CENTRELINE



(b) PAVEMENT REVOLVED ABOUT INNER EDGE



(c) PAVEMENT REVOLVED ABOUT OUTER EDGE

LEGEND

CROSS SECTION AT AA—NORMAL CAMBER

CROSS SECTION AT BB—ADVERSE CAMBER REMOVED

CROSS SECTION AT CC—SUPERELEVATION EQUAL TO CAMBER

CROSS SECTION AT DD—FULL SUPERELEVATION ACHIEVED

NOTE :—

THE RATE OF CHANGE OF SUPERELEVATION (LONGITUDINAL SLOPE OF EDGE COMPARED TO CENTRELINE) SHOULD BE MINIMUM 1 IN 150 FOR ROADS IN PLAIN AND ROLLING TERRAIN AND 1 IN 60 IN MOUNTAINOUS AND STEEP TERRAIN THE ACTUAL RATE USED WILL DETERMINE THE DISTANCES AB, BC AND CD

SCHEMATIC DIAGRAMS SHOWING DIFFERENT METHODS OF ATTAINING SUPERELEVATION