9. Discuss the scope of aerial surveys in preliminary survey for highway location. What are the steps to be followed ?
10. Explain how the final location and detailed survey of a highway are carried out.
11. Give the details of drawings to be prepared in highway project with the recommended scales and size of the drawings.
12. Explain briefly the various stages of work in a new Highway Project.
13. What are the conditions which necessitate taking up of a re-alignment project of the highway.
14. Discuss the general principles in the re-alignment of a highway and explain how the work is carried out.

Chapter 4

Highway Geometric Design

4.1 INTRODUCTION

4.1.1 Importance of Geometric Design

The geometric design of a highway deals with the dimensions and layout of visible features of the highway such as alignment, sight distances and intersections.

The geometrics of highway should be designed to provide optimum efficiency in traffic operations with maximum safety at reasonable cost. The designer may be exposed to either planning of new highway net work or improvement of existing highways to meet the requirements of the existing and the anticipated traffic.

It is possible to design and construct the pavement of a road in stages; but it is very expensive and rather difficult to improve the geometric elements of a road in stages at a later date. Therefore it is important to plan and design the geometric features of the road during the initial alignment itself taking into consideration the future growth of traffic flow and possibility of the road being upgraded to a higher category or to a higher design speed standard at a later stage.

Geometric design of highways deals with following elements :

(i) Cross section elements

(ii) Sight distance considerations

(iii) Horizontal alignment details

(iv) Vertical alignment details

(v) Intersection elements

Under cross section elements, the considerations for the width of pavement, formation and land, the surface characteristics and cross slope of pavement are included. The sight distance or clear distance visible ahead of a driver at horizontal and vertical curves and at intersections govern the safe movements of vehicles.

The change in the road directions are made possible by introducing horizontal curves. Super-elevation is provided by raising the outer edge of pavement to counteract the centrifugal force developed on a vehicle traversing a horizontal curve; extra pavement width is also provided on horizontal curves. In order to introduce the centrifugal force and the super-elevation gradually, transition curves are introduced between the straight and circular curves. The gradients and vertical curves are introduced in the vertical alignment of a highway. Design of road intersections with facilities for safe and efficient traffic movement needs adequate knowledge of traffic engineering.

Highway geometrics are greatly influenced by the topography, locality and traffic characteristics and the requirements of design speed. The factors which control the geometric design requirements are speed, road user and vehicular characteristics, design traffic, traffic capacity and benefit-cost considerations. However, speed is the factor which is important governing most of the geometric design elements of roads, as may be seen from the subsequent articles of this chapter.

4.1.2 Design Controls and Criteria

The geometric design of highways depends on several design factors. The important of these factors which control the geometric elements are :

(i) Design speed

(ii) Topography

(iii) Traffic factors

(iv) Design hourly volume and capacity

(v) Environmental and other factors.

(i) Design Speed

The design speed is the most important factor controlling the geometric design elements of highways. The design speed is decided taking into account the overall requirements of the highway. In India different speed standards have been assigned depending upon the importance or the class of the road such as National/State Highways, Major/Other District Roads and Village Roads. Further the design speed standards are modified depending upon the terrain or topography. Similarly urban roads have a different set of design speeds.

Design of almost every geometric design element of a road is dependent on the design speed. For example the requirements of the pavement surface characteristics, the cross section element of the road such as width and clearance requirements, the sight distance requirements, the horizontal alignment elements such as radius of curve super-elevation, transition curve length and the vertical alignments such as gradient, summit and valley curve lengths-all these depend mainly on the design speed of the road.

(ii) Topography

The topography or the terrain conditions influence the geometric design of highway significantly. The terrains are classified based on the general slope of the country across the alignment, as plain rolling, mountainous and steep terrains as given in Art. 4.4.2. The design standards specified for different classes of roads are different depending on the terrain classification. For example the design or ruling speed of NH and SH on plain terrain with general cross slope upto 10% is 100 kmph whereas the speed on rolling

terrain with general cross slope of 10 to 25% is 80 kmph and that on mountainous terrain with cross slope 25 to 60% is 50 kmph. As the speed standards affect every geometric design element, topography also affects the geometric design of roads. Further in hilly terrain, it is necessary to allow for steeper gradients and sharper horizontal curves due to the construction problems.

(iii) Traffic Factors

The factors associated with the traffic that affect geometric design of roads are the vehicular characteristics and human characteristics of road users. It is difficult to decide the design vehicle or the standard traffic lane under the mixed traffic flow condition prevalent especially on urban roads of developing countries. This is a complex problem. The different vehicle classes such as passenger cars, buses, trucks, motor cycles, etc. have different speed and acceleration characteristics, apart from having different dimensions and weights. However, it is often necessary to consider some standard vehicle as the design vehicle. The important human factors which affect traffic behaviour include the physical, mental and psychological characteristics of drivers and pedestrians.

(iv) Design Hourly Volume and Capacity

The traffic flow or volume keeps fluctuating with time, from a low value during off-peak hours to the highest value during the peak hour. It will be uneconomical to design the roadway facilities for the peak traffic flow or the highest hourly traffic volume. Therefore a reasonable value of traffic volume is decided for the design and this is called the design hourly volume. This value is to be determined from extensive traffic volume studies as discussed in Art. 5.2.3. The ratio of volume to capacity affects the level of service of the road.

(v) Environmental and Other Factors

The environmental factors such as aesthetics, landscaping, air pollution, noise pollution and other local conditions should be given due consideration in the design on road geometrics. Some of the arterial high speed highways and expressways are designed for higher speed standards and uninterrupted flow of vehicles by providing grade separated intersections and controlled access.

4.2 HIGHWAY CROSS SECTION ELEMENTS

4.2.1 Pavement Surface Characteristics

The pavement surface depends on the pavement type which is decided based on the availability of materials and funds, volume and composition of traffic, subgrade, and climatic conditions, construction facilities and cost considerations. The important surface characteristics of the pavement are the friction unevenness, light reflecting characteristics and drainage of surface water.

Friction

The friction between vehicle tyre and pavement surface is one of the factors determining the operating speed and distance requirements in stopping and accelerating the vehicles. When a vehicle negotiates a horizontal curve, the lateral friction developed counteracts the centrifugal force and thus governs the safe operating speed. Frictional force is an important factor in the acceleration and retardation abilities of vehicles. The coefficient of friction or the skid resistance offered by the pavement surface under various driving and surface conditions is important with reference to the safety. The maximum coefficient of friction comes into play only when the braking efficiency is high enough to partially arrest the rotation of the wheels on application of brakes, at low speeds.

Skid occurs when the slide without revolving or when the wheels partially revolve i.e., when the path travelled along the road surface is more than the circumferential movements of the wheels due to their rotation. When the brakes are applied, the wheels are locked partially or fully, and if the vehicle moves forward, the longitudinal skidding takes place which may vary from 0 to 100 percent.

While a vehicle negotiates a horizontal curve, if the centrifugal force is greater than the counteracting forces (i.e. lateral friction and component of gravity due to super-elevation) lateral skidding takes place. The lateral skid is considered dangerous as the vehicle goes out of control leading to an accident. The maximum lateral skid coefficient is generally equal to or slightly higher than the forward skid coefficient in braking tests.

Slip occurs when a wheel revolves more than the corresponding longitudinal movement along the roads. Slipping usually occurs in the driving wheel of a vehicle when the vehicle rapidly accelerates from stationary position or from slow speed on pavement surface which is either slippery and wet or when the road surface is loose with mud.

Factors affecting friction or skid resistance

The maximum friction offered by pavement surface or the skid resistance depends upon the following factors :

(i) Type of pavement surface namely, cement concrete bituminous, WBM, earth surface etc.

(ii) Macro-texture of the pavement surface or its relative roughness.

(iii) Condition of pavement namely, wet or dry, smoothened or rough, oil spilled, mud or dry sand on pavement.

(iv) Type and condition of tyre i.e. new with good treads or smoothened and worn out tyre.

(v) Speed of vehicle

(vi) Extent of brake application or brake efficiency

(vii) Load and tyre pressure

(viii) Temperature of tyre and pavement, and

(ix) Type of skid, if any

The type of aggregate used and the mix design of pavement surface course affect the skid resistance of the pavements, particularly in the case of old or worn out pavements.

The coefficient of friction reduces considerably when the pavement surface is smooth or wet. The coefficient of friction also decreases slightly with increase in temperature, tyre pressure and load. Smooth and worn out tyres offer higher friction factors on dry pavement then new tyres with treads because of large areas of contact. But on wet pavements new tyres with good treads give higher friction factors than worn out tyres.

This is because the lubricating effect of water on the wet pavement is reduced as the water entrapped between the tyre and pavement escapes into the treads of the tyre. Hence new tyres are more dependable than smooth ones in adverse surface and other conditions prone to skidding, such as wet pavements. The minimum anticipated value of coefficient of friction under worst possible pavement condition is generally taken for design purposes. The friction coefficient decreases with skid speed, which in turn depends on the speed of vehicle and brake efficiency.

For the calculation of stopping distance, the longitudinal friction coefficient values of 0.35 to 0.40 have been recommended by the Indian Roads Congress, depending upon speed. See Art. 4.3.2. These values have been suggested keeping in view the minimum coefficient of friction in the longitudinal direction on wet pavements and after allowing a suitable factor of safety. Further when a longitudinal friction coefficient of 0.40 is allowed for stopping the vehicle, the resultant retardation is 3.93 m/sec^2 which is not too uncomfortable to the passengers.

In the case of horizontal curve design, the Indian Roads Congress has recommended the lateral coefficient of friction of 0.15. This low value of transverse skid resistance has been suggested considering the worst possible surface condition such as mud on pavement surface at horizontal curve with super-elevation, during the rains, as it is essential to prevent possible lateral skid, even under adverse pavement conditions.

Pavement unevenness

Higher operating speeds are possible on even pavement surfaces with less undulations than on uneven and poor surfaces. Pavement surface should hence be maintained with minimum possible unevenness so that the desired speed can be maintained in conformity with other geometric standards. Pavement unevenness also affects vehicle operation cost, comfort and safety. Fuel consumption and wear and tear of tyres and other moving parts increases with increase in pavement unevenness. Loose road surfaces increase the tractive resistance and hence causes increase in fuel consumption. Uneven surfaces also increase fatigue and accidents.

The pavement surface condition is commonly measured by using an equipment called Bump Integrator, in terms of unevenness index, which is the cumulative measure of vertical undulations of the pavement surface recorded per unit horizontal length of the road. For example, unevenness index may be measured in cm per km. It has been shown from the tests that it is desirable to keep the unevenness index low, and preferably less than 150 cm/km for good pavement surfaces of high speed highways. A value of 250 cm/km is satisfactory upto a speed of about 100 kmph. Value more than 350 cm/km is considered very uncomfortable even at speed of 50 kmph. (See Art. 10.4.2 for evaluation of pavement surface condition). Pavement undulations are also some times measured using a straight edge in terms of the extent of number of depression or ruts along and across the pavement.

An Unevenness Indicator has been designed and patented by the Central Road Research Institute, New Delhi. This equipment is useful to indicate unevenness values from 3 to 20 mm.

The unevenness or undulations on pavement surface may be caused by various factors, such as : (i) inadequate or improper compaction of the fill, subgrade and pavement layers (ii) un-scientific construction practices including the use of boulder stones and bricks as soling course over loose subgrade soil (iii) use of inferior pavement materials (iv) improper surface and subsurface drainage (v) use of improper construction machinery (vi) poor maintenance practices and (vii) localized failures due to combination of causes.

Light reflecting characteristics

Night visibility very much depends upon the light reflecting characteristics of the pavement surface. The glare caused by the reflection of head lights is considerably high on wet pavement surface than on the dry pavement. Light colored or white pavement surface give good visibility at night particularly during rains, and they produces glare and eye strain during bright sunlight. Black top pavement surface on the other hand provides very poor visibility at nights, especially when the surface is wet.

4.2.2 Cross Slope or Camber

Cross slope or *chamber* is the slope provided to the road surface in the transverse direction to drain off the rain water from the road surface. Drainage and quick disposal of water from the pavement surface by providing cross slope is considered important because of two reasons :

(i) To prevent the entry of surface water into the subgrade soil through pavement; the stability, surface condition and the life of the pavement get adversely affected if the water enters in the subgrade and the soil gets soaked.

(ii) To prevent the entry of water into the bituminous pavement layers, as continued contact with water causes stripping of bitumen from the aggregates and results in deterioration of the pavement layer.

(iii) To remove the rain water from the pavement surface as quickly as possible and to allow the pavement to get dry soon after the rain; the skid resistance of the pavement gets considerably decreased under wet condition, rendering it slippery and unsafe for vehicle operation at high speeds.

Usually the camber is provided on the straight roads by raising the center of the carriageway with respect to the edges, forming a crown or highest point on the center line. At horizontal curves with super-elevation, the surface drainage is effected by raising the outer edge of pavement with respect to the inner edge while providing the desired super-elevation. The rate of camber or cross slope is usually designated by 1 in n which means that the transverse slope is in ratio 1 vertical to n horizontal. Camber is also expressed as a percentage. If the camber is x%, the cross slope is x in 100.

The required camber of a pavement depends on :

(i) the type of pavement surface, and

(ii) the amount of rainfall

A flat camber of 1.7 to 2.0% is sufficient on relatively impervious pavement surface like cement concrete or bituminous concrete. In pervious surface like water bound macadam or earth road which may allow surface water to get into the subgrade soil, steeper cross slope is required. Steeper camber are also provided in areas of heavy rainfall.

The minimum camber needed to drain off surface water may be adopted keeping in view the type of pavement surface and the amount of rainfall in the locality. Too steep cross slope is not desirable because of the following reasons :

(i) Transverse tilt of vehicles causes uncomfortable side thrust and a drag on the steering of automobiles. Also the thrust on the wheels along the pavement edges is more causing unequal wear of the tyres as well as road surface.

(ii) Discomfort causing throw of vehicle when crossing the crown during overtaking operations.

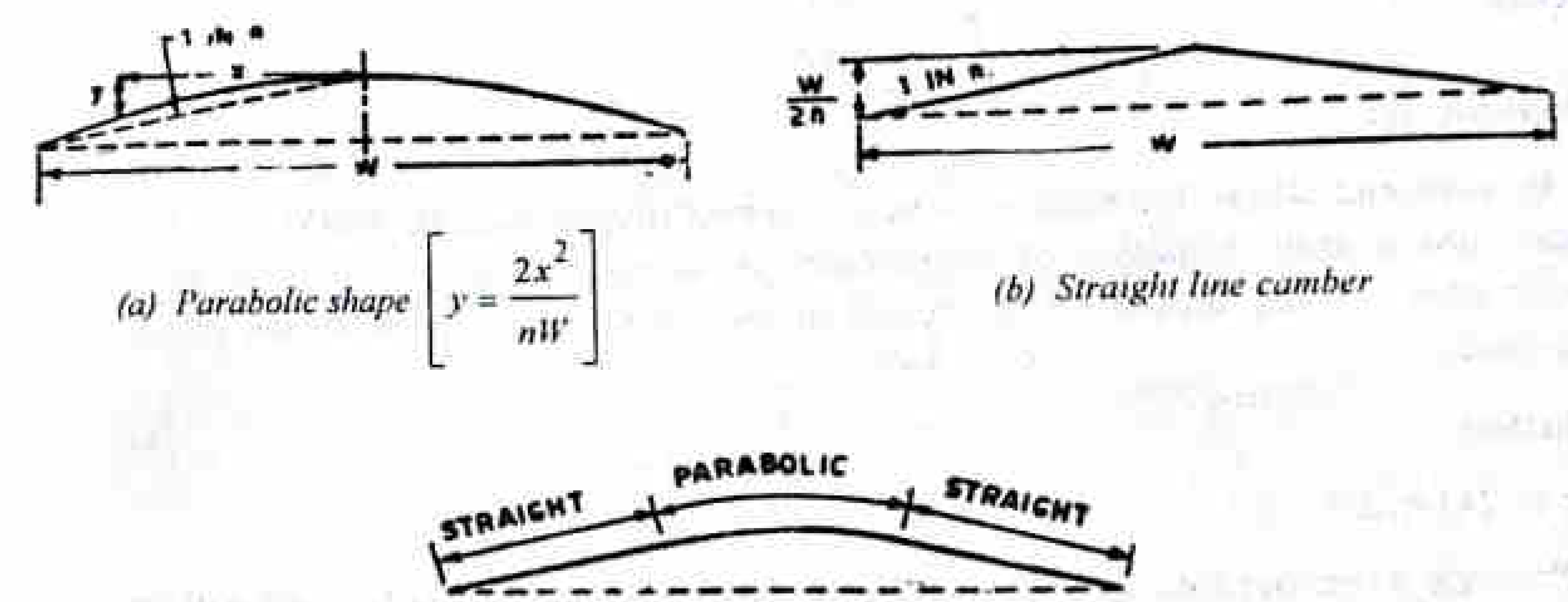
(iii) Problems of toppling over of highly laden bullock carts and trucks.

(iv) Formation of cross ruts due to rapid flow of water.

(v) Tendency of most of the vehicles to travel along the center line

Shape of cross slope

The camber is given a parabolic elliptic or straight line shape in the cross section. Parabolic or elliptic shape is given so that the profile is flat at the middle and steeper towards the edges, which is preferred by fast moving vehicles as they have to frequently cross the crown line during overtaking operation on a two lane highway. See Fig. 4.1a.



(a) Parabolic shape $\left[y = \dfrac{2x^2}{nW} \right]$ *(b) Straight line camber*

(c) Combination of straight and parabolic shape

Note : vertical scale are enlarged in the above sketches

Fig. 4.1 Shapes of Cross Slope

When very flat cross slope is provided as in cement concrete pavements, straight line shape of camber may be provided as shown in Fig. 4.1b. Steel tyred wheels of animal drawn vehicles can cause considerable damage to the pavement surface due to high stresses. The wheel does not have full contact increasing further the contact stress under these steel tyred wheels when the vehicle travels along the center of the pavement with straight camber. Some times a combined camber with parabolic central portion and straight line camber at the edges as shown in Fig. 4.1 c is preferred.

The values of camber recommended by the IRC for different types of road surfaces are given in table 4.1. A range of values are given with a view that in localities with lower rainfall, a flatter chamber and in places with high rainfall, a steeper camber can be adopted.

Table 4.1 Recommended values of camber for different types of road surfaces

Sl. No.	Types of road surface	Range of camber in areas of rainfall range		
		Heavy	to	Light
1.	Cement concrete and high type bituminous surface	1 in 50 (2.0 %)	to	1 in 60 (1.7 %)
2.	Thin bituminous surface	1 in 40 (2.5 %)	to	1 in 50 (2.0 %)
3.	Water bound macadam, and gravel pavement	1 in 33 (3.0 %)	to	1 in 40 (2.5 %)
4.	Earth	1 in 25 (4.0 %)	to	1 in 33 (3.0 %)

The cross slope for shoulders should be 0.5% steeper than the cross slope of adjoining pavement, subject to a minimum of 3.0% (and a maximum value of 5.0% for earth shoulders).

Providing camber in the field

For providing the desired amount and shape of camber, templates of camber boards are prepared with the specified camber. These are used to check the lateral profile of finished pavement during construction. Depending on the shape of the camber chosen, the camber board may be prepared. Forming a straight line camber is very simple. In the case of parabolic camber, the general equation $y = x^2/a$ may be adopted.

Here $a = nW/2$ for a pavement of width W and cross slope 1 in n.

Hence,
$$y = \frac{2x^2}{nW} \tag{4.1}$$

Example 4.1

In a district where the rainfall is heavy, major district road of WBM pavement, 3.8 m wide, and a state highway of bituminous concrete pavement, 7.0 m wide are to be constructed. What should be the height of the crown with respect to the edges in these two cases ?

Solution

For WBM road

Provide a camber rate of 1 in 33 as the rainfall is heavy. Rise of crown with respect to edges

$$= \frac{3.8}{2} \times \frac{1}{33} = 0.058 \text{ m}$$

For bituminous concrete road

Provide a cross fall of 1 in 50.

Rise of crown with respect to the edges

$$= \frac{7}{2} \times \frac{1}{50} = 0.07 \text{ m}$$

4.2.3 Width of Pavement or Carriageway

The pavement or carriageway width depends on the width of traffic lane and number of lanes. The carriageway intended for one line of traffic movement may be called a traffic lane. The lane width is determined on the basis of the width of vehicle and the minimum side clearance which may be provided for the safety. When the side clearance is increased (up to a certain limit) there is an increase in operating speed of vehicles and hence an increase in capacity of the traffic lane. Keeping all these in view a width of 3.75 m is considered desirable for a road having single lane for vehicles of maximum width 2.44 m. For pavements having two or more lanes, width of 3.5 m per lane is considered sufficient.

The maximum width of vehicle as per IRC specifications is 2.44 m. For details refer Art. 5.2. If a single lane carriageway of width 3.8 m is provided, a side clearance of

0.68 m would be obtained as shown in Fig. 4.2a. In the case of two-lane pavement of width 0.7 m, a minimum clearance between two lanes of traffic would be 1.06 m for the widest vehicles on the road, as shown in Fig. 4.2b.

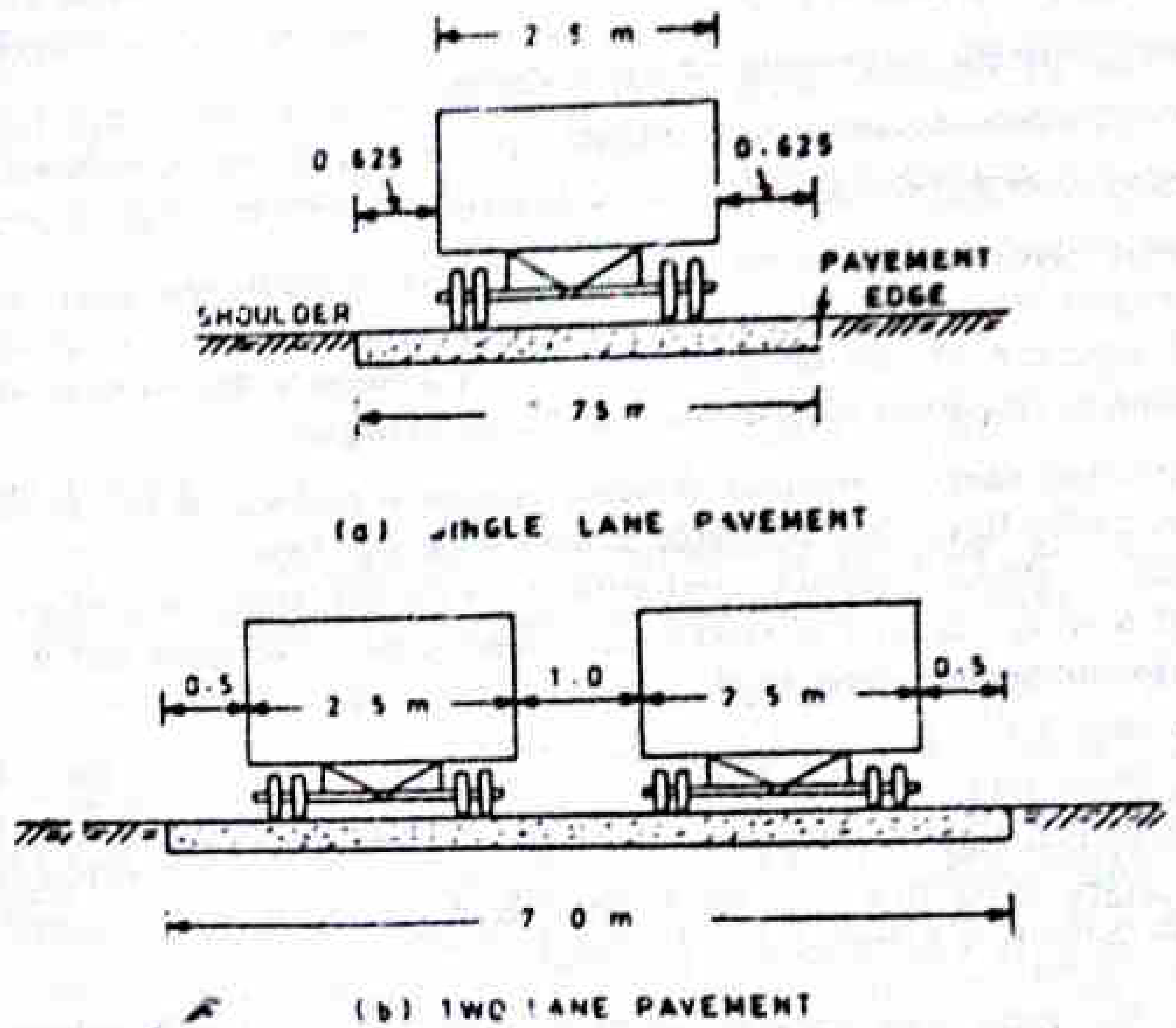


Fig. 4.2 Lateral Placement of Vehicles

The number of lanes required in a highway depends on the predicted traffic volume and the design traffic volume of each lane. The width of pavement is increased on horizontal curves as discussed in Art. 4.3.5.

In some highways, *traffic separators or medians* are provided between two sets of traffic lanes intended to divide the traffic moving in opposite directions. In such highways the road width depends on the pavement width (or the lane widths and number of lanes) and the width of traffic separators. The width of carriageway for various classes of roads standardised by Indian Roads Congress are given in Table 4.2.

Table 4.2 Width of Carriageway

	Class of road	Width of carriageway
(i)	Single lane	3.75 m
(ii)	Two lanes, without raised kerbs	7.0 m
(iii)	Two lanes, with raised kerbs	7.5 m
(iv)	Intermediate carriageway (except on important roads)	5.5 m
(v)	Multi-lane pavements	3.5 m per lane

Notes : (i) The width of single lane or village roads may be decreased to 3.0 m

(ii) On urban roads without kerbs the single lane width may be decreased to 3.5 m and in access roads to residential areas to 3.0 m

(iii) The minimum width recommended for kerbed urban road is 5.5 m to make allowance for a stalled vehicle.

Traffic separators of medians

The main function of traffic separator is to prevent head-on collision between vehicles moving in opposite directions on adjacent lanes.

The separators may also help to

(i) Channelize traffic into streams at intersections

(ii) shadow the crossing and turning traffic

(iii) segregate slow traffic and to protect pedestrians.

The traffic separators used may be in the form of pavement markings, physical dividers or area separators. Pavement marking is the simplest of all these. The mechanical separator should be designed in such a manner that even if wheels of a vehicle encroach, no part of vehicle body should be damaged.

Area separators may be medians, dividing islands or parkway strips, dividing the two directions of traffic flow. It is desirable to have wide area separators of 8 to 14 m width. But the width should be decided in conformity with the availability of land and its cost. A minimum of 6 m is required to reduce head light glare.' The glare can be reduced in narrower strips by planting trees or shrubs.

The IRC recommends a minimum desirable width of 5.0 m for medians of rural highways, which may be reduced to 3.0 m where land is restricted. On long bridges the width of the median may be reduced upto 1.2 to 1.5 m. The medians should normally be of uniform width on a particular road, but where change in width is unavoidable, a transition of 1 in 15 to 1 in 20 must be provided.

On urban highways with six lanes or more, medians should invariably be provided. The minimum recommended width of medians at intersections of urban roads are 1.2 m for pedestrian refuge, 4.0 to 7.5 m for protection of vehicles making right turn and 9.0 to 12 m for protection vehicles crossing at grade. The absolute minimum width of median in urban area is 1.2 m and desirable minimum is 5.0 m.

4.2.4 Kerbs

Kerb indicates the boundary between the pavement and shoulder; or sometimes islands or foot path or kerb parking space. It is desirable to provide kerbs on urban roads (See Fig. 4.3). There are a variety of kerb designs. Kerbs may be mainly divided into three groups based on their functions.

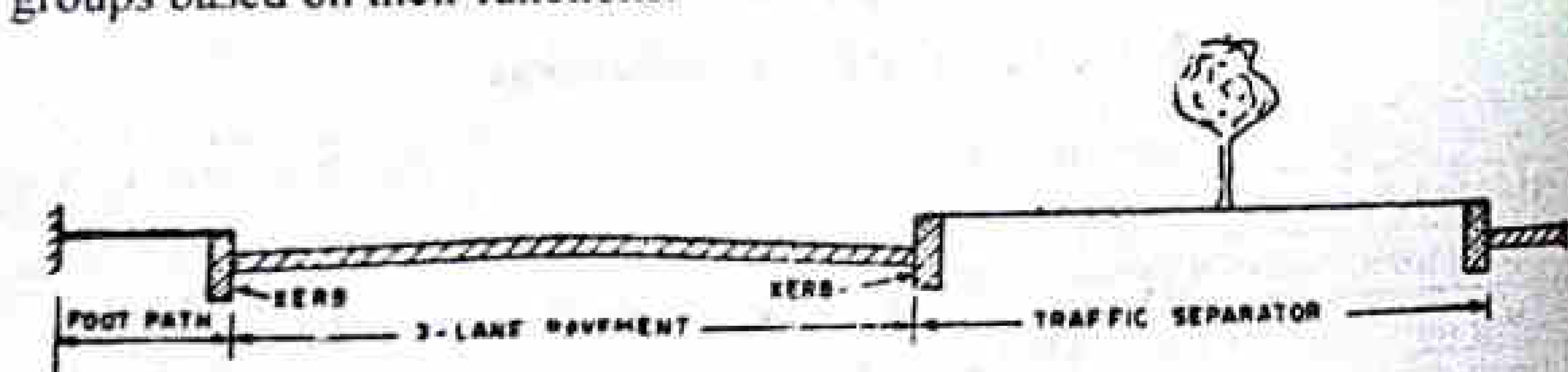


Fig. 4.3 Kerb and Traffic Separator

(i) *Low or mountable type kerbs* which though encourage traffic to remain in the through traffic lanes, yet allow the driver to enter the shoulder area with little difficulty. The height of this type of shoulder kerbs is about 10 cm above the pavement edge with a slope or batter to help vehicles climb the kerb easily. This type of kerb is provided at medians and channelization schemes and is also useful for longitudinal drainage system.

(ii) *Semi-barrier type kerb* is provided on the periphery of a roadway where the pedestrian traffic is high. This type of kerb has a height of about 15 cm above the pavement edge with a batter of 1 : 1 on the top 7.5 cm. This kerb prevents encroachment of parking vehicles, but at acute emergency it is possible to drive over this kerb with some difficulty.

(iii) Barrier type kerb is provided in built-up areas adjacent to foot paths with considerable pedestrian traffic. The height of kerb stone is about 20 cm above the pavement edge with a steep batter of 1.0 vertical 0.25 horizontal.

In rural roads submerged kerbs are sometimes provided at pavement edge between edge and shoulders. These kerbs provide lateral confinement and stability to the granular base course and flexible pavements.

4.2.5 Road Margins

The various elements included in the road margins are shoulder, parking lane, frontage road, driveway, cycle track, footpath, guard rail and embankment slope.

Shoulders are provided along the road edge to serve as an emergency lane for vehicle compelled to be taken out of the pavement or roadway. Shoulders also act as service lanes for vehicles that have broken down. Refer Fig. 4.4, which gives cross section details of roads in embankment and cutting. The width of shoulder should be adequate to accommodate stationary vehicle fairly away from the edge of adjacent lane. It is desirable to have a minimum shoulder width of 4.6 m so that a truck stationed at the side of the shoulder would have a clearance of 1.85 m from the pavement edge. The minimum shoulder width recommended by the IRC is 2.5 m.

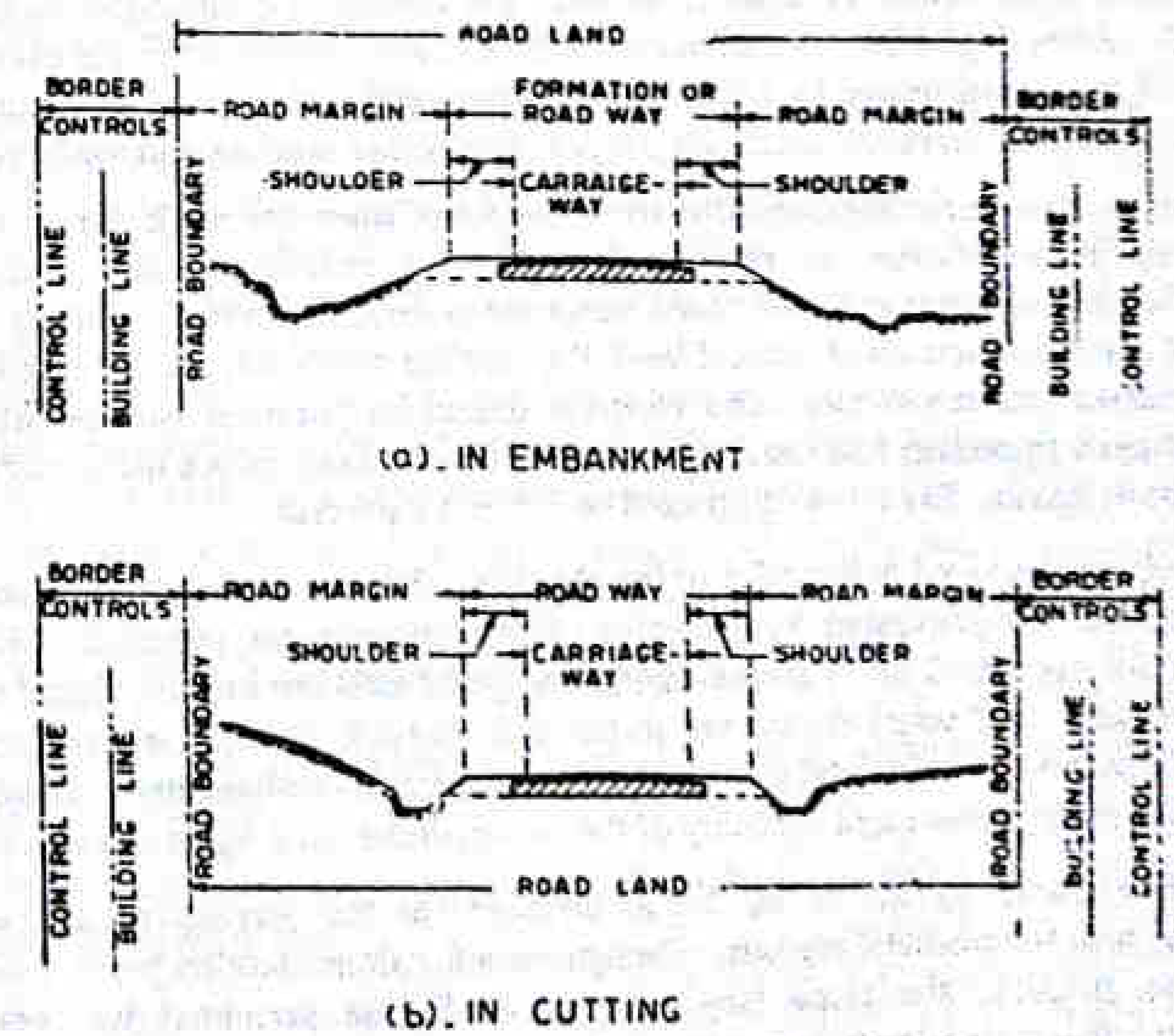


Fig. 4.4 Cross Section Details

The shoulders should have sufficient load bearing capacity to support loaded truck even in wet weather. The surface of the shoulder should be rougher than the traffic lanes so that vehicles are discouraged to use the shoulder as a regular traffic lane. The colour of the shoulder should preferably be different from that of the pavement so as to be distinct.

Parking lanes are provided on urban roads to allow kerb parking. As far as possible only *parallel parking* should be allowed as it is safer for moving vehicles. Also the clearance available between the parked vehicles and the edge of adjacent lane is more in the case of parallel parking than in *angle parking*. The parking lane should also have sufficient width; 3.0 m width is required for parallel parking.

Lay-byes are provided near public conveniences with guide maps to enable drivers to stop clear off the carriageway. Lay-byes should normally be of 3.0 width and atleast 30 m length with 15 m end tapers on both sides.

Bus bays may be provided by recessing the kerb to avoid conflict with moving traffic. Bus bays should be located atleast 75 m away from the intersections.

Frontage roads are provided to give access to properties along an important highway with controlled access to express way or free way. The frontage roads may run parallel to the highway and are isolated by a separator, with approaches to the through facility only at selected points, preferably with grade separations.

Drive ways connect the highway with commercial establishment like fuel-stations, service-stations etc. Drive ways should be properly designed and located, fairly away from an intersection. The radius of the drive way curve should be kept as large as possible, but the width of the drive way should be minimised to reduce the length of cross walks.

Cycle tracks are provided in urban areas when the volume of cycle traffic on the road is very high. Refer Fig. 4.10. A minimum width of 2 m is provided for the cycle track and the width may be increased by 1.0 m for each additional cycle lane. The layout of the cycle tracks should be carefully decided in large highway intersections and traffic rotaries.

Footpath or side walks are provided in urban areas when the vehicular as well as pedestrian traffic are heavy, to provide protection to pedestrians and to decrease accidents. See Fig. 4.3, 4.9 and 4.10. Side walks are generally provided on either side of the road and the minimum width should be 1.5 m and the width may be increased based on the pedestrian traffic volume. The footpath should be provided with a surface as smooth as or even smoother than the adjacent traffic lane so as to induce the pedestrian to keep on to the footpath. The cross fall should be 2.5 to 3.0 percent.

Guard rails are provided at the edge of the shoulder when the road is constructed on a fill so that vehicles are prevented from running off the embankment, especially when the height of the fill exceeds 3 m. Various designs of guard rails are in use. Guard stones (painted with black and white strips) are installed at suitable intervals along the outer edge of the formation at horizontal curves of roads running on embankments along rural areas so as to provide better night visibility of the curves under head lights of vehicles.

Embankment slopes should be as flat as possible for the purpose of safe traffic movement and also for aesthetic reasons. Though from the slope stability point, a steeper slope may be possible, the slope may be kept as flat as permitted by economic considerations. Road side landscaping can improve the aesthetic features of road side, making road travel more pleasant.

4.2.6 Width of Roadway or Formation

Width of formation or roadway is the sum of widths of pavements or carriageway including separators if any; and the shoulders. Formation or roadway width is the top width of the highway embankment or the bottom width of highway cutting excluding the side drains, as shown in Fig. 4.4. The width of roadway standardized by the Indian Roads Congress are given in Table 4.3.

Table 4.3 Width of Roadway of various classes of roads

Sl. No.	Road classification	Roadway width, m at :	
		Plain and Rolling terrain	Mountainous and Steep terrain
1.	National & State Highways (a) Single lane (b) Two lane	 12.0 12.0	 6.25 8.80
2.	Major District Roads (a) Single lane (b) Two lanes	 9.0 9.0	 4.75 -
3.	Other District Roads (a) Single (b) Two lanes	 7.5 9.0	 4.75 -
4.	Village roads-single lane	7.5	4.00

Notes (i) In multilane highways, roadway width should be adequate for the requisite number of traffic lanes besides shoulders and central median.

(ii) The minimum roadway width on single lane bridge is 4.25 m.

4.2.7 Right of Way

Right of way is the area of land acquired for the road, along its alignment. The width of this acquired land is known as *land width* and it depends on the importance of the road and possible future development. A minimum land width has been prescribed for each category of road. A desirable range of land width has also been suggested for each category. While acquiring land for a highway it is desirable to acquire more width of land as the cost of adjoining land invariably increases very much, soon after the new highway is constructed. Also road side developments start taking place making it difficult later on to acquire more land if required for future widening or for other improvements. In some cases the lower width within the suggested range may have to be adopted in view of high cost of land and other existing features. This is particularly true in urban and industrial areas.

The land width is governed by the following factors :

(i) Width of formation depending on the category of highway and width of roadway and road margins.

(ii) Height of embankment or depth of cutting which is governed by the topography and the vertical alignment.

(iii) Side slopes of embankment or cutting which depend on the height of the slope, soil type and several other considerations including aesthetics.

(iv) Drainage systems and their size, which depends on the rainfall, topography, and run off.

(v) Sight distance considerations on horizontal curves, as there is restriction to the visibility on the inner side of the curve due to obstruction such as building structures etc. At sharp curves it is desirable to acquire a wider strip of land in order to avoid obstructions to visibility. Refer Fig. 4.5.

(vi) Reserve land for future widening is to be planned in advance based on anticipated future development and increase in the traffic.

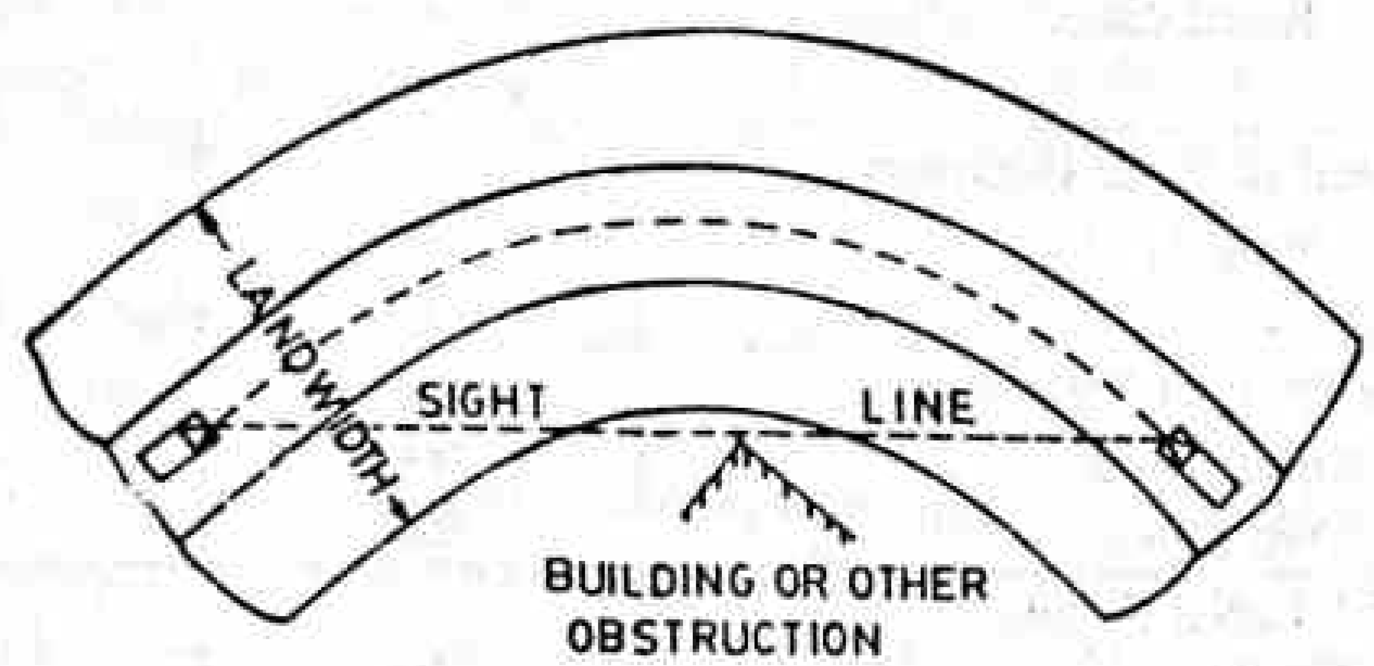


Fig. 4.5 Obstructions to Visibility at Horizontal Curve

The values of normal and range of land width standardized by the IRC for various categories of roads in rural areas and in different terrains are given in Table 4.4 (a).

It is desirable to control the building construction activities on either side of the road boundary, beyond the land width acquired for the road, in order to reserve sufficient space for future improvement of roads. Therefore, it is necessary to disallow the building activities upto "building line" with sufficient setback from the road boundary. In addition, it is desirable so exercise control of the nature of building upto further set back distance upto the "control lines". The overall width requirements between the building lanes and between the control lines on either side of the road, recommended by the IRC for different classification of roads in rural areas at different terrain conditions are given in Table 4.4 (b). It may be seen from Tables 4.4 (a) and 4.4 (b) that the normal land width required for the National and State Highways on open plain terrain is 45 m and the maximum land width required is 60 m, the corresponding width between the building lines is 80 m and that between the control lines is 150 m, thus allowing set back distances of 10 and 45 m beyond the road boundary lines with the maximum recommended road width.

Table 4.4 (a) Recommended land width for different classes of rural roads (metre)

Sl. No.	Road classification	Plain and rolling terrain				Mountainous and steep terrain	
		Open areas		Built-up areas		Open areas	Built-up areas
		Normal	Range	Normal	Range	Normal	Normal
1.	National and State Highways	45	30-60	30	30-60	24	20
2.	Major District Roads	25	25-30	20	15-25	18	15
3.	Other District Roads	15	15-5	15	15-20	15	12
4.	Village Roads	12	12-18	10	10-15	9	9

Table 4.4 (b) Recommended standards for building lines and control lines

Road Classification	Plain and Rolling terrain			Mountainous & steep terrain	
	Open area		Built-up area	Distance between building line and road boundary (set back), m	
	Overall width between building lines, m	Overall width between control lines, m	Distance between building line and road boundary (set-back), m	Open areas	Built-up areas
N.H. & S.H.	80	150	3 to 6	3 to 5	3 to 5
M.D.R.	50	100	3 to 5	3 to 5	3 to 5
O.D.R	25/30*	35	3 to 5	3 to 5	3 to 5
V.R.	25	30	3 to 5	3 to 5	3 to 5

Note : *If the land width is equal to the width between building lines indicated in this column, the building lines should be set back 2.5 m from the road land boundary.

The recommended land widths for different classes of urban roads are, 50 to 60 m for arterial roads (high types of urban roads meant for through traffic, with controlled access), 30 to 40 m for sub-arterial roads, 20 to 30 m for collector streets (urban roads and streets meant for collecting traffic from local streets and feed to the arterial and sub-arterial roads) and 10 to 20 m for local streets.

4.2.8 Typical Cross Sections of Roads

Some of the typical cross sections of rural roads of different categories and urban roads are shown in Fig. 4.6 to 4.10.

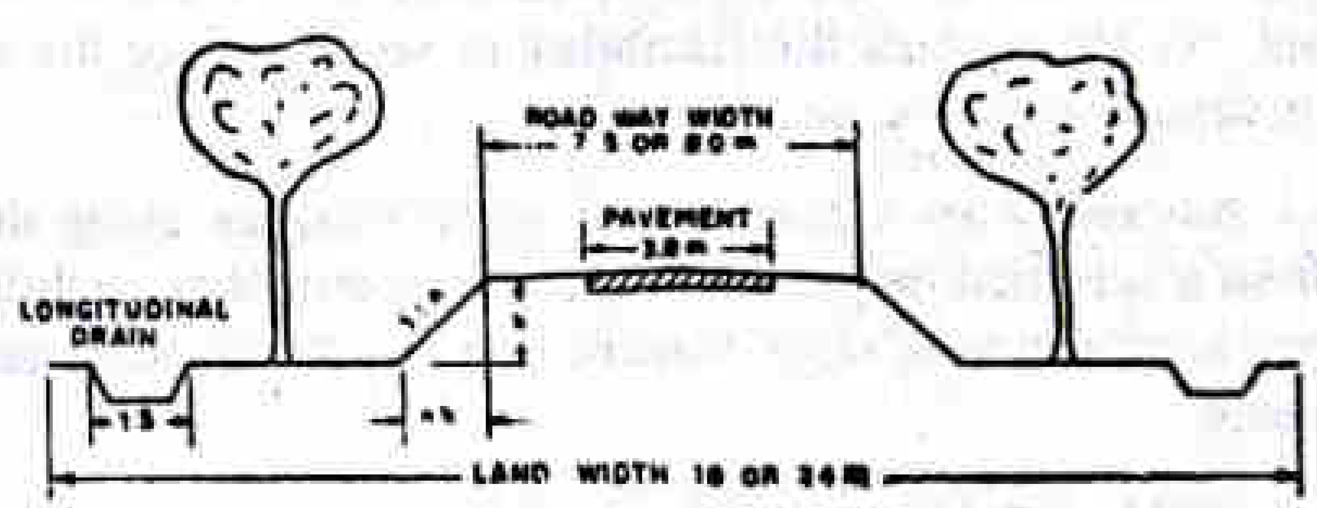


Fig. 4.6 Cross Section of VR or ODR in Embankment in Rural Area

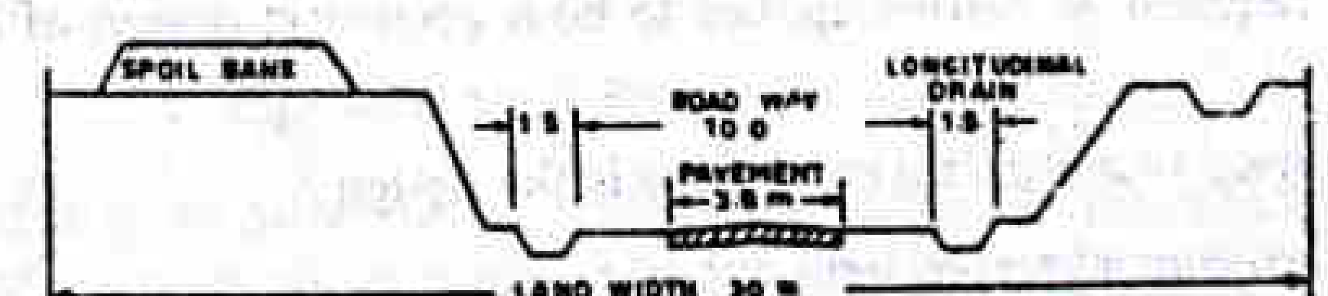


Fig. 4.7 Cross Section of Major District Road in Cutting in Rural Area

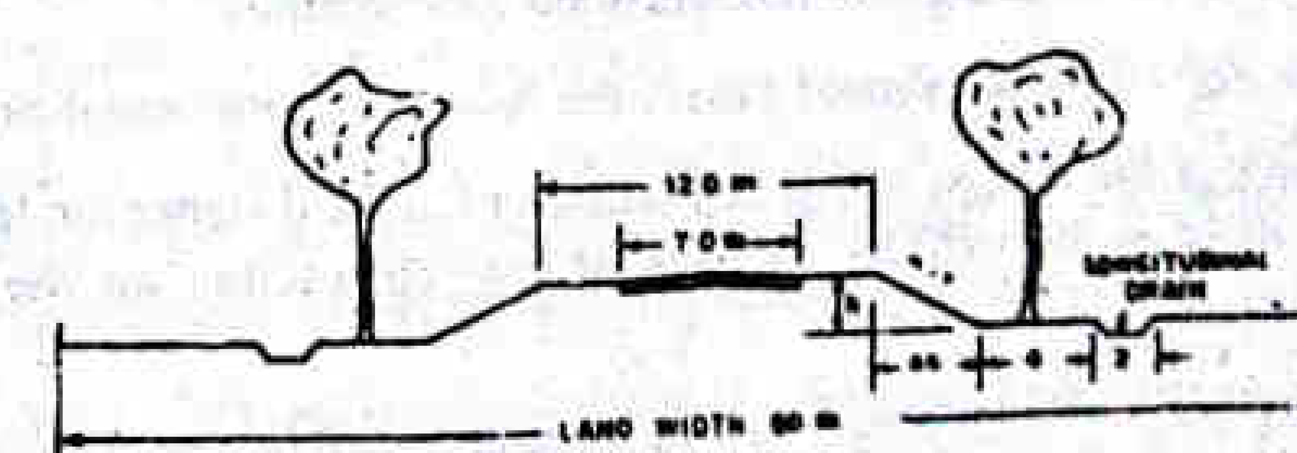


Fig. 4.8 Cross Section of National or SH in Rural Area

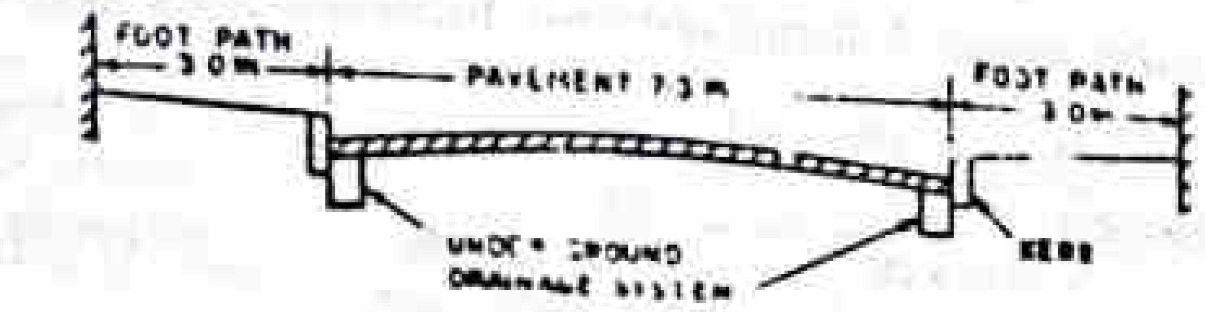


Fig. 4.9 Cross Section of Two-lane City Road in Built-up Area

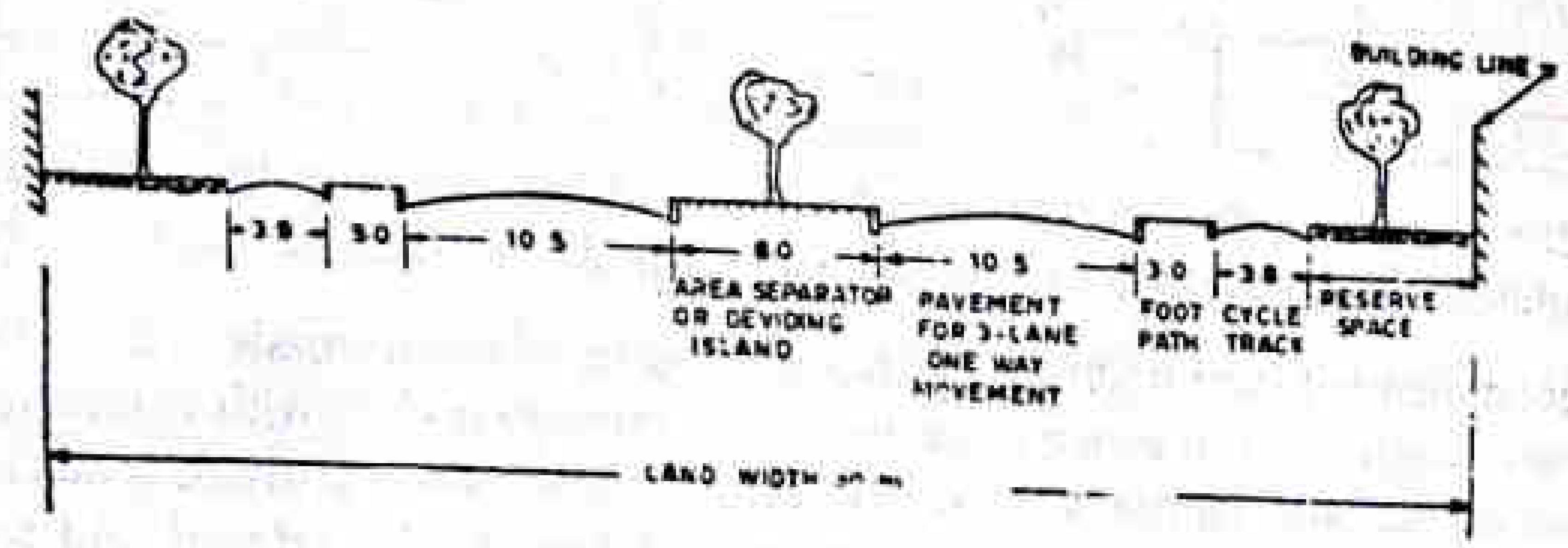


Fig. 4.10 Cross Section of Divided Highway in Urban Area

4.3 SIGHT DISTANCE

4.3.1 Introduction

The safe and efficient operation of vehicle on roads depends, among other factor on the road length at which an obstruction, if any, becomes visible to the driver in the direction of travel. In other words the feasibility to see ahead, or the visibility is very important for safe vehicle operation on a highway.

Sight distance available from a point is the actual distance along the road surface, which a driver from a specified height above the carriageway has visibility of stationary or moving objects. In other words, sight distance is the length of road visible ahead to the driver at any instance.

Restrictions to sight distance may be caused at horizontal curves, by objects obstructing vision at the inner side of the road or at vertical summit curves or at intersections. These are shown in Fig. 4.11.

Sight distance required by drivers applies to both geometric design of highways and for traffic control.

Three sight distance situations are considered in the design :

(i) Stopping or absolute minimum sight distance

(ii) Safe overtaking or passing sight distance, and

(iii) Safe sight distance for entering into uncontrolled intersections

The standards for sight distance should satisfy the following three conditions :

(i) Driver travelling at the design speed has sufficient sight distance or length of road visible ahead to stop the vehicle, in case of any obstruction on the road ahead, without collision.

(ii) Driver traveling at the design speed should be able to safely overtake, at reasonable intervals, the slower vehicles without causing obstruction or hazard to traffic of opposite direction.

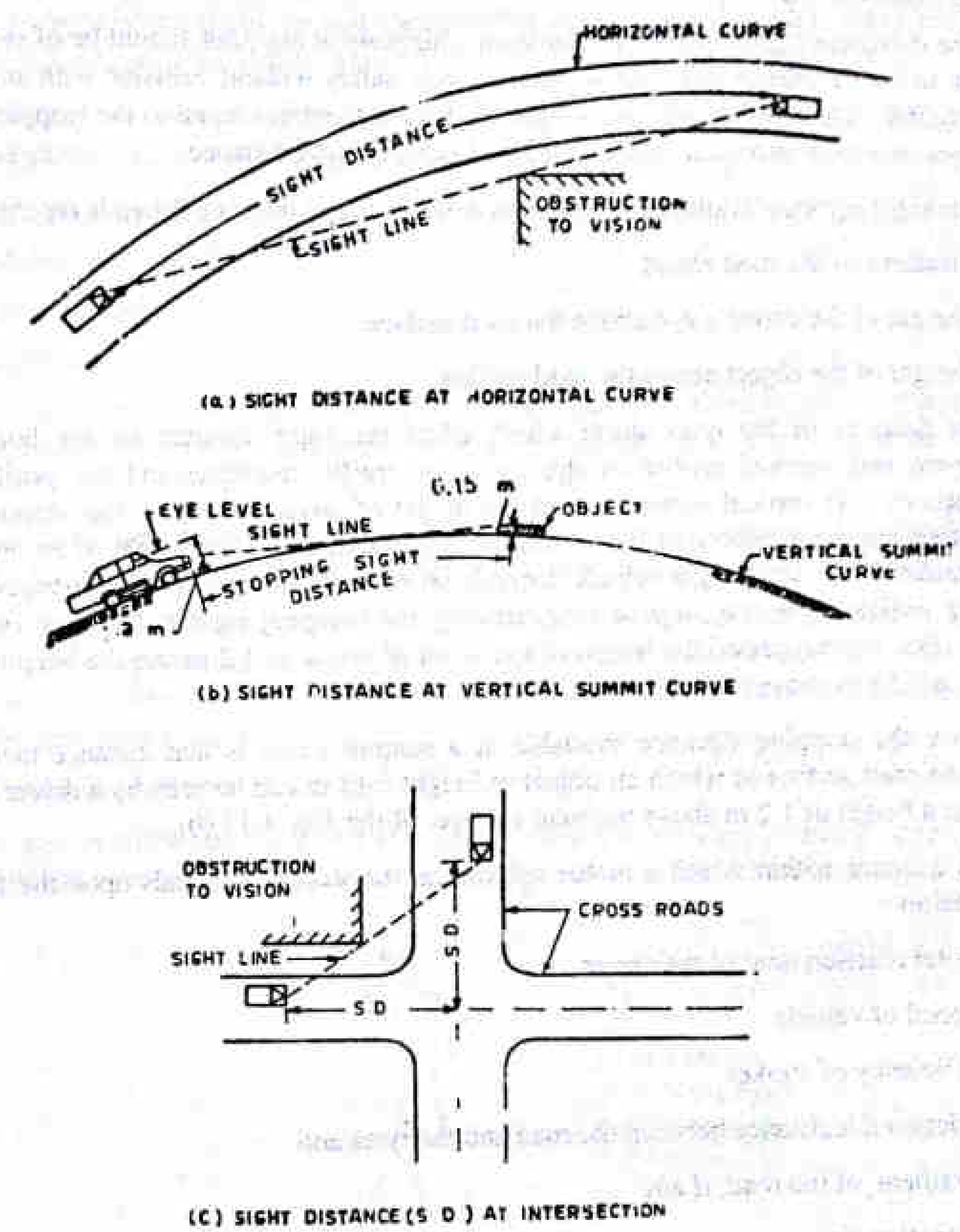


Fig. 4.11 Sight Distance Consideration

(iii) Driver entering an uncontrolled intersection (particularly unsignalised intersection) has sufficient visibility to enable him to take control of his vehicle and to avoid collision with another vehicle.

Apart from the three situations mentioned above, the following sight distances are considered by the IRC in highway design :

(i) *Intermediate sight distance* – This is defined as twice the stopping sight distance. When overtaking sight distance can not be provided, intermediate sight distance is provided to give limited overtaking opportunities to fast vehicles.

(ii) *Head light sight distance* – This is the distance visible to a driver during night driving under the illumination of the vehicle head lights. This sight distance is critical at up-gradients and at the ascending stretch of the valley curves.

4.3.2 Stopping Sight Distance (SSD)

The minimum sight distance available on a highway at any spot should be of sufficient length to stop a vehicle traveling at design speed, safely without collision with any other obstruction. The absolute minimum sight distance is therefore equal to the stopping sight distance, which is also some times called *non-passing* sight distance.

The sight distance available on a road to a driver at any instance depends on

(i) features of the road ahead,

(ii) height of the driver's eye above the road surface.

(iii) height of the object above the road surface.

The features of the road ahead which affect the sight distance are the horizontal alignment and vertical profile of the road, the traffic condition and the position of obstructions. At vertical summit curves the height of driver's eye and the object above road level are more important factors affecting the visibility. The height of an object to be considered for stopping a vehicle depends on what might be a source of danger to the moving vehicle. For the purpose of measuring the stopping sight distance or visibility ahead. IRC has suggested the height of eye level of driver as 1.2 m and the height of the object as 0.15 m above the road surface.

Hence the stopping distance available at a summit curve is that distance measured along the road surface at which an object of height 0.15 m can be seen by a driver where eye is at a height of 1.2 m above the road surface. Refer Fig. 4.11 (b).

The distance within which a motor vehicle can be stopped depends upon the factors listed below :

(a) Total reaction time of the driver

(b) Speed of vehicle

(c) Efficiency of brakes

(d) Frictional resistance between the road and the tyres and

(e) Gradient of the road, if any

Total reaction time

Reaction time of the driver is the time taken from the instant the object is visible to the driver to the instant the brakes are effectively applied. The amount of time gap depends on several factors. During this time the vehicle travels a certain distance at the original speed or the design speed. Thus the stopping distance increases with increase in reaction time of the driver. The total reaction time may be split up into two parts.

(i) perception time

(ii) brake reaction time

The *perception time* is the time required for a driver to realise that brakes must be applied It is the time from the instant the object comes on the line of sight of the driver to the instant he realises that the vehicle needs to be stopped. The perception time varies from driver to driver and also depends on several other factors such as speed of the vehicle, distance of object and other environmental conditions.

The *brake reaction time* also depends on several factors including the skill of the driver, the type of the problems and various other environmental factors. Often the total brake reaction time of the driver is taken together.

PIEV Theory : According to this theory the total reaction time of the driver is split into four parts, viz., time taken by the driver for :

(i) Perception

(ii) Intellection

(iii) Emotion, and

(iv) Volition

Perception time is the time required for the sensations received by the eyes or ears to be transmitted to the brain through the nervous system and spinal chord. In other words, it is the time required to perceive an object or situation.

Intellection time is the time required for understanding the situation. It is also the time required for comparing the different thoughts, regrouping and registering new sensations.

Emotion time is the time elapsed during emotional sensations and disturbance such as fear, anger or any other emotional feelings such as superstition etc. with reference to the situation. Therefore the emotion time of a driver is likely to vary considerably depending upon the problems involved.

Volition time is the time taken for the final action.

It is also possible that the driver may apply brakes or take any avoiding action by the reflex action, even without thinking. The PIEV process has been illustrated in Fig. 4.12.

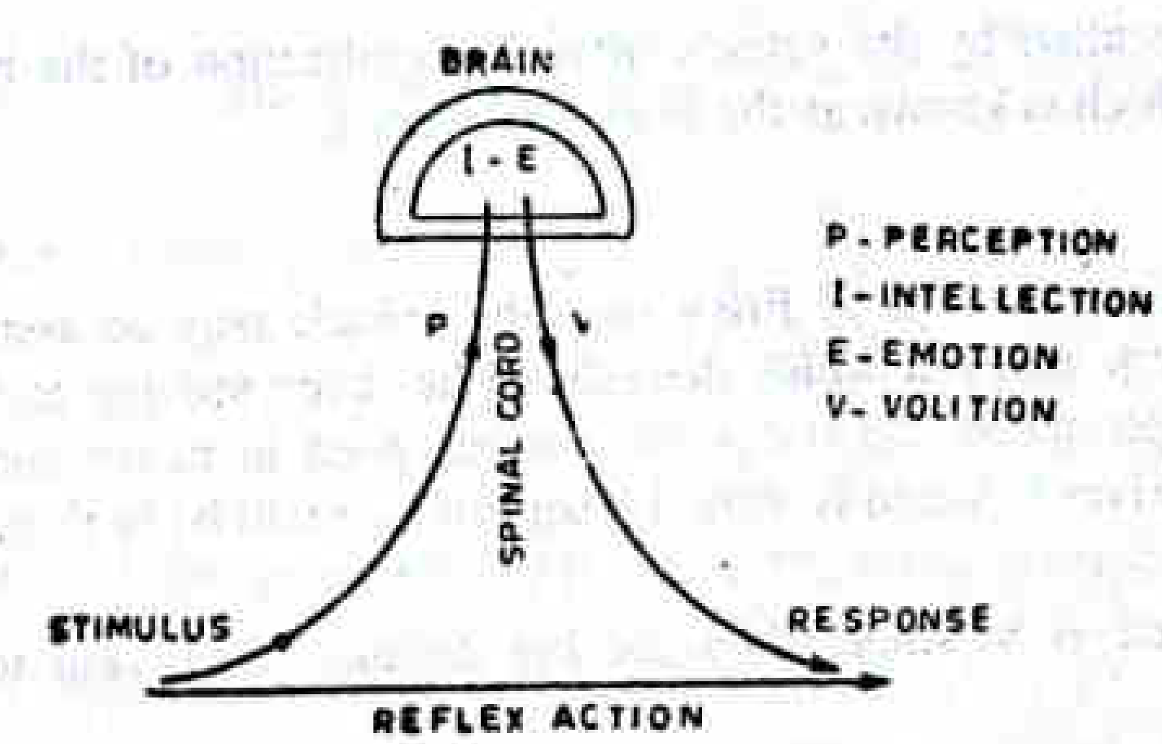


Fig. 4.12 Reaction Time and PIEV Process

The PIEV time of a driver depends on several factors such as physical and psychological characteristics of the driver, type of the problem involved, environmental condition and temporary factors (e.g., motive of the trip, travel speed, fatigue consumption of alcohol, etc.). The total reaction time of an average driver may vary from 0.5 second for simple situations to as much as 3 to 4 seconds or even more in complex problems.

Speed of vehicle

The stopping distance depends very much on the speed of the vehicle. First, during the total reaction time of the driver the distance moved by the vehicle will depend on the speed. Second, the braking distance or the distance moved by the vehicle after applying the brakes, before coming to a stop depends also on the initial speed of the vehicle. Hence it is evident that higher the speed, higher will be the stopping distance.

Efficiency of brakes

The braking efficiency is said to be 100 percent if the wheels are fully locked preventing them from rotating on application of the brakes. This will result in 100 percent skidding which is normally undesirable, except in utmost emergency. Also skidding is considered to be dangerous, as it is not possible to control a skidding vehicle. Hence to avoid skid, the braking forces should not exceed the frictional force between the wheels and tyres.

Frictional resistance between road and tyres

The frictional resistance developed between road and tyres or the *skid resistance* depends on the type and condition of the road surface and the tyres as discussed in article 4.11. The braking distance increases with decrease in skid resistance. IRC has specified a design friction coefficient of 0.35 to 0.4 depending upon the speed to be used for finding the braking distance in the calculation of stopping sight distance. This value, apart from having sufficient safety factor, permits a rate of retardation which is fairly comfortable for passengers.

Analysis of stopping distance

The stopping distance of a vehicle is the sum of :

(i) the distance travelled by the vehicle during the total reaction time know as *lag distance* and

(ii) the distance travelled by the vehicle after the application of the brakes, to a dead stop position which is known as the *braking distance.*

Lag distance

During the total reaction time or PIEV time the vehicle may be assumed to proceed forward with a uniform speed at which the vehicle has been moving and this speed may be taken as the design speed. If 'v' is the design speed in m/sec and 't' is the total reaction time of the driver in seconds, then the lag distance will be 'v.t' metres.

If the design speed is V kmph, then the lag distance works out to $V \times \frac{1000}{60 \times 60}$ t = 0.278 V.t meters.

The total reaction time of driver depends on a variety of factors and a value of 2.5 secs. Is considered reasonable for most situations. The IRC has also recommended the value of reaction time t = 2.5 secs. for the calculation of stopping distance.

Braking distance

The coefficient of friction 'f' depends on several factors such as the type and condition of the pavement surface and tyres. Also the value of f decreases with increase in speed. IRC recommends the following f-values for design :

Speed, kmph	20 to 30	40	50	60	65	80	100
Longitudinal coefficient of friction, f	0.40	0.38	0.37	0.36	0.36	0.35	0.35

Assuming a level road, the braking distance may be obtained by equating the work done in stopping the vehicle and the kinetic energy.

If F is the maximum frictional force developed and the braking distance is l, then work done against friction force in stopping the vehicle is $F \times l = f\,W\,l$, where W is the total weight of the vehicle.

The kinetic energy at the design speed of v m/sec will be

$$\frac{1}{2} m v^2 = \frac{W v^2}{2g}$$

Hence
$$f W l = \frac{W v^2}{2g}$$

or
$$l = \frac{v^2}{2 g f}$$

Here l = braking distance, m

v = speed of vehicle, m/sec.

f = design coefficient of friction

= 0.4 to 0.35 depending on speed, from 30 to 80 kmph

g = acceleration due to gravity = 9.8 m/sec^2.

Stopping distance = lag distance + braking distance

i.e.,
$$\text{SD, m} = vt + \frac{v^2}{2gf} \tag{4.1}$$

If speed is V kmph, stopping distance

$$\text{SD, m} = \left[0.278\,V_t + \frac{V^2}{254 f}\right] \tag{4.2}$$

Equation 4.1 and 4.2 are the general equations for stopping distance at level.

Stopping distance at slopes

When there is an ascending gradient of say, + n% the component of gravity adds to the braking action and hence the braking distance is decreased. The component of gravity acting parallel to the surface which adds to the braking force is equal to W sin α = W tan α = Wn/100.

Equating kinetic energy and work done,

$$\left(fW + \frac{Wn}{100}\right) l = \frac{1}{2}\frac{W v^2}{g}$$

$$l = \frac{v^2}{2g\left(f + \frac{n}{100}\right)}$$

Similarly, in descending gradient of – n% the braking distance increases, as the component of gravity now opposes the braking force. Hence the equation is given by :

$$\left(fW + \frac{Wn}{100}\right) l = \frac{W v^2}{2g}$$

$$l = \frac{v^2}{2g\left(f - \frac{n}{100}\right)}$$

Hence the general equation 4.1 for stopping distance may now be modified for n% gradient and may be written as :

$$\text{SD, m} = \left[vt + \frac{v^2}{2g\,(f \pm 0.01\,n)}\right] \qquad (4.3)$$

When the ground is level, n = 0 Eq. 4.3 reduces to Eq. 4.1.

Equation 4.3 may be re-written as follows where the speed is V kmph and the gradient is n percent :

$$\text{SD, m} = 0.278\ V.t + \frac{V^2}{254\,(f \pm 0.01)\,n} \qquad (4.4)$$

As the Stopping Sight Distance SSD required on descending gradient is higher, it is necessary to determine the critical value of the SSD for the descending gradient on the roads with gradients and two way traffic flow.

The minimum stopping sight distance hence should be equal to the stopping distance in one-way traffic lanes and also in two-way traffic roads when there are two or more traffic lanes. On roads with restricted width or on single lane roads when two-way movement of traffic is permitted, the minimum stopping sight distance should be equal to *TWICE* the stopping distance to enable both vehicle coming from opposite directions to stop. The SSD should invariably be provided throughout the length of all roads and hence this is also known as *absolute minimum sight distance*. When the stopping sight distance for the design speed is not available on any section of a road, the speed should be restricted by a warning sign and a suitable speed-limit regulation sign. However this should be considered only as a temporary measure and wherever possible, the stretch of the road should be re-aligned or the obstruction to visibility removed so as to provide atleast stopping sight distance for the design speed.

The safe stopping distance values calculated in the similar manner for various design speeds and recommended by IRC are given in Table 4.5.

Table 4.5 Stopping sight distance values for different speeds

Design speed, kmph	20	25	30	40	50	60	65	80	100
Safe stopping sight distance for design, m	20	25	30	45	60	80	90	120	180

Example 4.2

Calculate the safe stopping sight distance for design speed of 50 kmph for (a) two-way traffic on a two lane road (b) two way traffic on a single plane road.

Assume coefficient of friction as 0.37 and reaction time of driver as 2.5 seconds

Solution

Stopping distance (Eq. 4.4) = lag distance + braking distance

$$= vt + \frac{v^2}{2gf}$$

$$V = 50 \text{ kmph or } v = \frac{50}{3.6} = 13.9 \text{ m/sec}$$

$$t = 2.5,\ g = 9.8,\ f = 0.37$$

$$\text{Stopping distance} = 13.9 \times 2.5 + \frac{13.9^2}{2 \times 9.8 \times 0.37}$$

$$= 34.8 + 26.6 = 61.4 \text{ m}$$

Alternatively, the stopping distance may also be calculated from Eq. 4.2 as follows :

$$SD = 0.278\ V_t + V^2/254\ f$$

$$= 0.278 \times 50 \times 2.5 + \frac{50^2}{254 \times 0.37} = 61.4 \text{ m}$$

(a) Stopping sight distance when there are two lanes = stopping distance = 61.4 m

(b) Stopping sight distance for two-way traffic with single lane = 2 [stopping distance] = 2 × 61.4 122.8.

Example 4.3

Calculate the minimum sight distance required to avoid a head-on collision of two cars approaching from the opposite directions at 90 and 60 kmph. Assume a reaction time of 2.5 seconds, coefficient of friction of 0.7 and a brake efficiency of 50 percent, in either case.

Solution

Stopping distance for one of the cars (Eq. 4.1).

$$\text{SD metres} = vt + \frac{v^2}{2gf}$$

$$V_1 = 90 \text{ kmph, } v = \frac{90}{3.6} = 25 \text{ m/sec}$$

$$V_2 = 60 \text{ kmph, } v = \frac{60}{3.6} = 16.67 \text{ m/sec}$$

As the brake efficiency is 50%, the wheels will skid through 50% of the braking distance and rotate through the remaining distance. Therefore, the value of coefficient of friction developed f may be taken as 50% of the coefficient of friction, i.e., f = 0.5 × 0.7 = 0.35.

The stopping distance for the first car SD_1

$$= 25 \times 2.5 + \frac{25^2}{2 \times 9.8 \times 0.35} = 153.6 \text{ m}$$

For second car, $$SD_2 = 16.67 \times 2.5 + \frac{16.67^2}{2 \times 9.8 \times 0.35} = 82.2 \text{ m}$$

Sight distance to avoid head-on collision of the two approaching cars

$$= SD_1 + SD_2 = 153.6 + 82.2 = 235.8 \text{ m.}$$

Example 4.4

Calculating the stopping sight distance on a highway at a descending gradient of 2% for a design speed of 80 kmph. Assume other data as per IRC recommendations.

Solution

Total reaction time t may be taken as 2.5 seconds and design coefficient of friction as f = 0.35.

$$V = 80 \text{ kmph}; n = -2\% = -0.02, G = 9.8 \text{ m/sec}^2$$

$$v = \frac{80}{3.6} = 22.2 \text{ m/sec}$$

SSD on road with gradient is given in Eq. 4.3 and 4.4.

From Eq. 4.3, $$SSD = vt + \frac{v^2}{2g(f \pm n\%)} = 2.2 \times 2.5 + \frac{22.2^2}{2 \times 9.8(0.35 - 0.02)}$$

$$= 55.5 + 76.2 = 131.7 \text{ m say } 132 \text{ m}$$

Alternatively, using Eq. 4.4

$$SSD = 0.278\, V.t + \frac{V^2}{254(f \pm 0.01)n}$$

$$= 0.278 \times 80 \times 2.5 + \frac{80^2}{254(0.35 - 0.02)} = 55.6 + 76.4 = 132 \text{ m}$$

Example 4.5

Calculate the values of (i) Head light sight distance and (ii) Intermediate sight distance for a highway with a design speed of 65 kmph. Assume suitably all the data required.

Solution

$$V = 65 \text{ kmph};$$

Assume $$f = 0.36, t = 2.5 \text{ secs.}$$

ight distance $$= SSD = 0.278\, Vt + \frac{V^2}{254 f}$$

$$= 0.278 \times 65 \times 2.5 + \frac{65^2}{254 \times 0.36} = 91.4 \text{ m}$$

(ii) Intermediate sight distance $= 2\text{ SSD} = 2 \times 91.4 = 182.8$ m

4.3.3 Overtaking Sight Distance (OSD)

If all the vehicles travel on a road at the design speed, then theoretically there should be no need for any overtaking. In fact all vehicles do not move at the designed speed and this is particularly true under mixed traffic conditions. In such circumstances, it is necessary for fast moving vehicles to overtake or pass the slow moving vehicles. It may not be possible to provide the facility to overtake slow moving vehicles throughout the length of a road. In such cases facilities for overtaking slow vehicles with adequate safety should be made possible at frequent distance intervals.

The minimum distance open to the vision of the driver of a vehicle intending to overtake slow vehicle ahead with safety against the traffic of opposite direction is known as the *minimum overtaking sight distance* (OSD) or the *safe passing sight distance* available.

The overtaking sight distance or OSD is the distance measured along the center of the road which a driver with his eye level 1.2 m above the road surface can see the top of an object 1.2 m above the road surface. Refer Fig. 4.13.

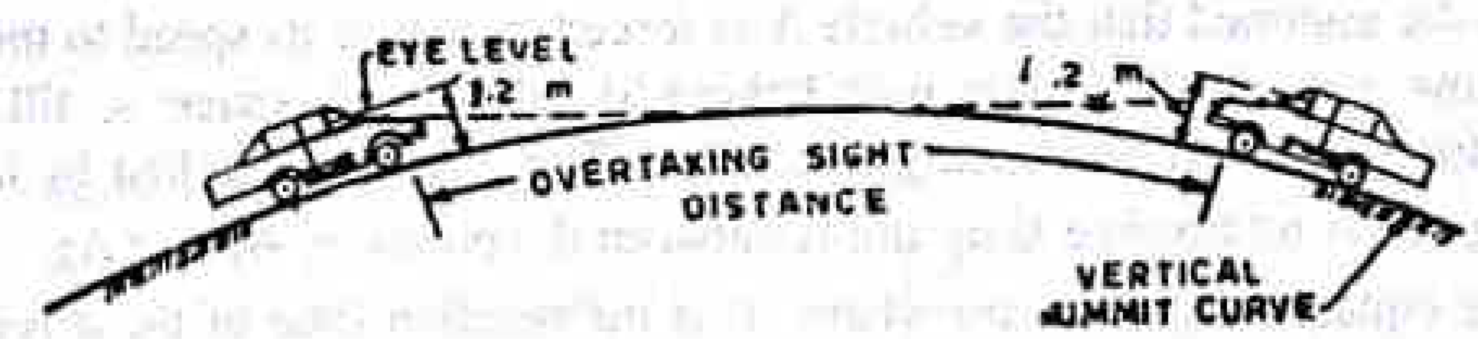


Fig. 4.13 Measurement of Overtaking Sight Distance

Some of the important factors on which the minimum overtaking sight distance required for the safe overtaking manoeuvre depends, are :

(a) speeds of (i) overtaking vehicle (ii) overtaken vehicle and (iii) the vehicle coming from opposite direction, if any.

(b) distance between the overtaking and overtaken vehicles; the minimum spacing depends on the speeds.

(c) skill and reaction time of the driver

(d) rate of acceleration of overtaking vehicle

(e) gradient of the road, if any

Analysis of Overtaking Sight Distance

Figure 4.14 shows the overtaking manoeuvre of vehicle A traveling at design speed, and another slow vehicle B on a two-lane road with two-way traffic. Third vehicle C comes from the opposite direction. The overtaking manoeuvre may be split up into three operations, thus dividing the overtaking sight distance into three parts, d_1, d_2 and d_3.

(i) d_1 is the distance travelled by overtaking vehicle A during the reaction time t sec of the driver from position A_1 to A_2.

(ii) d_2 is the distance travelled by the vehicle A from A_2 to A_3 during the actual overtaking operation, in time T sec.

(iii) d_3 is the distance travelled by on-coming vehicle C from C_1 to C_2 during the overtaking operation of A, i.e. T secs.

Certain assumptions are made in order to calculate the values of d_1, d_2 and d_3.

In Fig. 4.14, A is the overtaking vehicle originally traveling at design speed v m/sec or V kmph; B is the overtaken or slow moving vehicle moving with uniform speed v_b m/sec or V_b kmph; C is a vehicle coming from opposite direction at the design speed v m/sec or V kmph. In a two-lane road the opportunity to overtake depends on the frequency of vehicles from the direction and the overtaking sight distance available at any instant.

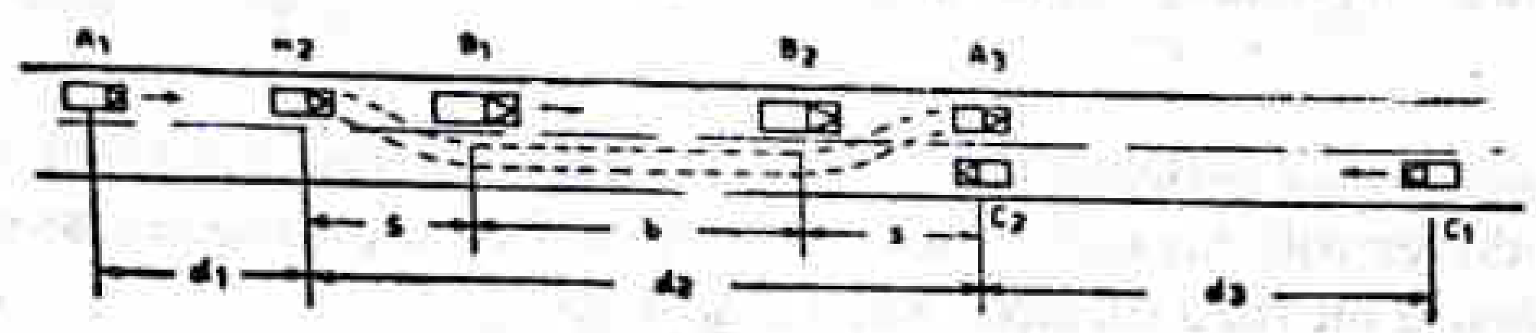


Fig. 4.14 Overtaking Manoeuvre

(i) It may be assumed that the vehicle A is forced to reduce its speed to the speed v_b of the slow vehicle B and moves behind it allowing a space s, till there is an opportunity for safe overtaking operation. The distance travelled by the vehicle A during this reaction time is d_1 and is between the positions A_1 and A_2. This distance will be equal to $v_b \times t$ metre where 't' is the reaction time of the driver in second. This reaction time 't' of the driver may be taken as two seconds as an average value, as the aim of the driver is only to find an opportunity to overtake. Thus,

$$d_1 = v_b\, t = 2\, v_b,\ m$$

(ii) From position A_2, the vehicle A starts accelerating, shifts to the adjoining lane, overtakes the vehicle B, and shifts back to it original lane ahead of B in position A_3 in time T sec. The straight distance between position A_2 and A_3 is taken as d_2. The minimum distance between position A_2 and B_1 may be taken as the minimum spacing 's' of the two vehicles while moving with the speed v_b m/sec. The minimum spacing between vehicles depends on their speed and is given by empirical formula :

$$s = (0.7\, v_b + 6),\ m$$

The minimum distance between B_2 and A_3 may also be assumed equal to s as mentioned above. If the time taken by vehicle A for the overtaking operation from position A_2 to A_3 is T second, the distance covered by the slow vehicle B traveling at a speed of v_b m/sec. = b = v_b/T m.

Now the time T depends on speed of overtaken vehicle B and the acceleration of overtaking vehicle A. This time T may be calculated by equating the distance d_2 to $(v_b\, T + ½\, a\, T^2)$, using the general formula for the distance travelled by an uniformly accelerating body with initial speed v_b m/sec and 'a' is the acceleration in m/sec^2.

$$d_2 = (b + 2s) = \left(v_b\, T + \frac{aT^2}{2}\right)$$

$$b = vb.T, \text{ and therefore } 2s = \frac{aT^2}{2}$$

Therefore, $$T = \sqrt{\frac{4s}{a}} \text{ sec, where } s = (0.7\, v_b + 6)$$

Hence, $$d_2 = (v_b\, T + 2s),\ m$$

(iii) The distance travelled by vehicle C moving at design speed v m/sec during the overtaking operation of vehicle A i.e. during time T is the distance d_2 between positions C_1 to C_2.

Hence, $$d_3 = v \times T$$

Thus the overtaking sight distance

$$OSD = (d_1 + d_2 + d_3)$$

$$= (v_b\, t + v_b\, T + 2s + vT) \qquad (4.5)$$

In kmph units, equations (4.5) worksout as :

$$OSD = 0.28\, V_b\, t + 0.28\, V_b\, T + 2s + 0.28\, V.T \qquad (4.6)$$

Here V_b = speed of overtaking vehicle, kmph

t = reaction time of driver = 2 secs.

V = speed of overtaking vehicle or design speed, kmph

$$T = \sqrt{\frac{4 \times 3.6s}{A}} = \sqrt{\frac{14.4s}{A}}$$

s = spacing of vehicles = $(0.2\, V_b + 6)$

A = acceleration, kmph/sec.

In case the speed of overtaken vehicle V_b is not given, the same may be assumed as (V – 16) kmph where V is the design speed in kmph or $v_b = (v - 4.5)$ m/sec and v is the design speed in m/sec.

The acceleration of the overtaking vehicle is to be specified. Usually this depends on the make of the vehicle, its condition, load and the speed. As a general guide Table 4.6 may be used for finding the maximum acceleration of vehicles at different speeds. The average rate of acceleration during overtaking manoeuvre may be taken corresponding to the design speed.

Table 4.6 Maximum overtaking acceleration at different speeds

Speed		Maximum overtaking acceleration	
V, kmph	v, m/sec	A, kmph/sec	a, m/sec^2
25	6.93	5.00	1.41
30	8.34	4.80	1.30
40	11.10	4.45	1.24
50	13.86	4.00	1.11
65	18.00	3.28	0.92
80	22.20	2.56	0.72
100	27.80	1.92	0.53

At overtaking sections, the minimum overtaking distance should be ($d_1 + d_2 + d_3$) when two-way traffic exists. On divide highways and on roads with one way traffic regulation, the overtaking distance need be only ($d_1 + d_2$) as no vehicle is expected from the opposite direction. On divided highways with four or more lanes, IRC suggests that it is not necessary to provide the usual OSD; however the sight distance on any highway should be more than the SSD, which is the absolute minimum sight distance.

Effect of grade in overtaking sight distance

Appreciable grades on the road, both the descending as well as ascending, increase the sight distance required for safe overtaking. In down grades though it is easier for the overtaking vehicles to accelerate and pass the overtaken vehicle may also accelerate and cover a greater distance 'b' during the overtaking time.

On up grades, the acceleration of the overtaking vehicle will be less and hence passing will be difficult; but the overtaken vehicle like heavily loaded trucks may also decelerate at steep ascends and compensate to some extent the passing sight distance requirement. Therefore the OSD at both ascending and descending grades are taken as equal to that at level stretch. However, at grades the overtaking sight distance should be greater than the minimum overtaking distance required at level.

The IRC has specified the safe values of overtaking sight distance required for various design speeds between 40 and 100 kmph. These values have been suggested based on the observation that 9 to 14 seconds are required by the overtaking vehicle for the actual overtaking manoeuvre depending on the design speed. This overtaking time may be increased by about two-third to take into account the distance covered by the vehicle from the opposing direction in the case of two-way traffic road, during the overtaking operation. The OSD values thus obtained for various design speeds are rounded off by the IRC and the recommended values of OSD on two lane highways are given in Table 4.7.

Table 4.7 Overtaking sight distance on two-lane highways for various speeds

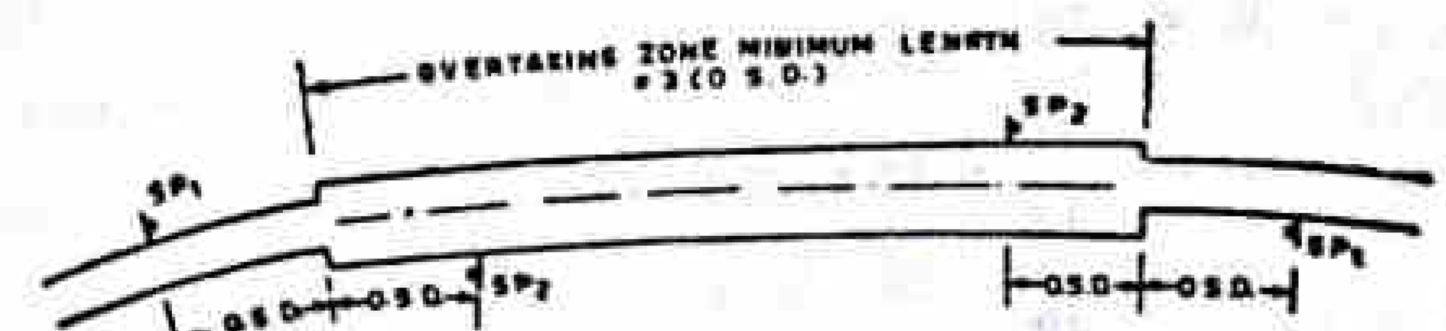
Speed kmph	Time component, seconds			Safe overtaking sight distance (metres)
	For overtaking manoeuvre	For opposing vehicle	Total	
40	9.0	6.0	15	165
50	10.0	7.0	17	235
60	10.8	7.2	18	300
65	11.5	7.5	19	340
80	12.5	8.5	21	470
100	14.0	9.0	23	640

Overtaking Zones

It is desirable to construct highways in such a way that the length of road visible ahead at every point is sufficient for safe overtaking. This is seldom practicable and there may be stretches where the safe overtaking distance can not be provided. In such zones where overtaking or passing is not safe or is not possible, sign posts should be installed indicating "*No Passing*" or "*Overtaking Prohibited*" before such restricted zones starts. But the overtaking opportunity for vehicles moving at design speed should be given at frequent intervals. These zones which are meant for overtaking are called *overtaking zones*.

The OSD and pavement width should be sufficient for safe overtaking operations. Sign posts should be installed at sufficient distance in advance to indicate the start of the overtaking zones; this distance may be equal to ($d_1 + d_2$) for one-way roads and ($d_1 + d_2 + d_3$) for two-way roads. Similarly the end of the overtaking zones should also be indicated by appropriate sign posts installed ahead at distances specified above. The minimum length of overtaking zone should be three time the safe overtaking distance i.e., 3 ($d_1 + d_2$) for one-way roads and 3 ($d_1 + d_2 + d_3$) for two-way roads. It is desirable that the length of overtaking zones is kept *five* times the overtaking sight distance.

Figure 4.15 shows an overtaking zone with specifications for the positions of the sign posts.



O.S.D = Overtaking sight distance
= ($d_1 + d_2$) for one way traffic
= ($d_1 + d_2 + d_3$) for two way traffic
SP$_1$ = Sign post "over taking zone ahead"
SP$_2$ = Sign post "end of overtaking zone"

Fig. 4.15 Overtaking Zones

Criteria for Sight Distance Requirements on Highway

The absolute minimum sight distance required throughout the length of the road is the SSD which should invariably be provided at all places. On horizontal curves the obstruction on the inner side of the curves should be cleared to provide the required set back distance and absolute minimum sight distance. The common obstruction to clear vision on horizontal curves are buildings and other structures, trees, advertisement boards, cut slopes, etc. On vertical summit curves the sight distance requirement may be fulfilled by proper design of the vertical alignment as given in Article 4.5. At uncontrolled intersections sufficient clearances to the sight lines may be given to provide for SSD.

Intermediate Sight Distance

Sufficient overtaking sight distance should be available on most of the road stretches. On horizontal curves the overtaking sight distance requirements can not always be fulfilled especially on sharp curves, if the safe overtaking sight distance requirements are high. In such cases overtaking should be prohibited by regulatory signs. In case of vertical summit curves, it is possible to provide the sight distance requirements by suitably designing the vertical alignment. At stretches of the road where required overtaking sight distance can not be provided as per Table 4.7, as far as possible *Intermediate Sight Distance*, ISD equal to twice SSD may be provided. (Refer Table 4.5). The measurement of the ISD may be made assuming both the height of the eye level of the driver and the object to be 1.2 metres above the road surface.

Example 4.6

The speed of overtaking and overtaken vehicles are 70 and 40 kmph, respectively on a two way traffic road. If the acceleration of overtaking vehicle is 0.99 m/sec^2.

(a) calculate safe overtaking sight distance

(b) mention the minimum length of overtaking zone and

(c) draw a neat-sketch of the overtaking zone and show the positions of the sign posts.

Solution

(a) Overtaking sight distance for two way traffic

$$= d_1 + d_2 = d_3 \qquad (4.5)$$

Assume the design speed as the speed of overtaking vehicle A

$$V = 70 \text{ kmph}$$

$$v = \frac{70}{3.6} = 19.4 \text{ m/sec}$$

$$v_b = \frac{40}{3.6} = 11.1 \text{ m/sec}$$

Acceleration, $a = 0.99$ m/sec per sec.

$$D_1 = v_b.\, t \text{ (Adopt } t = 2 \text{ secs)} = 11.1 \times 2 = 22.2 \text{ m}$$

$$d_2 = v_b.T + 2.s$$

$$s = (0.7\, v_b + 6) = (0.7 \times 11.1 + 6) = 13.8 \text{ m}$$

$$T = \sqrt{\frac{4.s}{a}} = \sqrt{\frac{4 \times 13.8}{0.99}} = 7.47 \text{ secs}$$

$$d_2 = 11.1 \times 7.47 + 2 \times 13.8 = 110.5 \text{ m}$$

$$d_3 = v.\, T = 19.4 \times 7.47 = 144.9 \text{ m}$$

$$\text{O.S.D} = d_1 + d_2 + d_3$$

$$= 22.2 + 110.5 + 144.9 = 277.6 \text{ m, say } 278 \text{ m}$$

(b) Minimum length of overtaking zone = 3 (OSD)

$= 3\,(d_1 + d_2 + d_3)$ for two-way traffic $= 3 \times 278 = 834$ metres

Desirable length of overtaking zone = 5 × (OSD) = 5 × 278 = 1390 m

(c) The details of the overtaking zone are shown in Fig. 4.16.

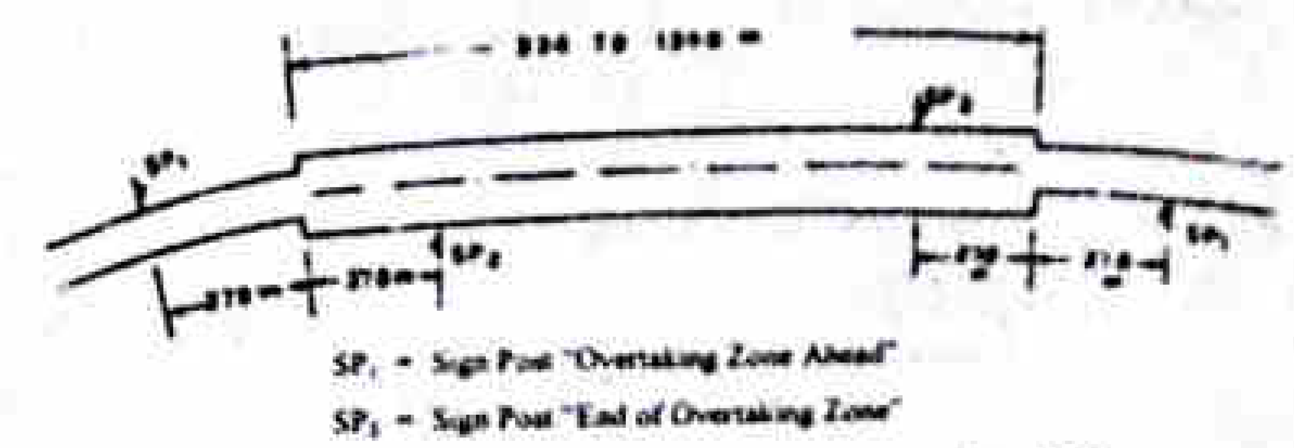


Fig. 4.16 Overtaking Zone (Example 4.6)

Example 4.7

Calculate the safe overtaking sight distance for a design speed of 96 kmph. Assume all other data suitably.

Solution

$$\text{O.S.D} = (d_1 + d_2) \text{ for one-way traffic}$$

$$= (d_1 + d_2 + d_3) \text{ for two-way traffic}$$

$$V = 96 \text{ kmph}$$

Assume $V_b = V - 16 = 80$ kmph and

A 2.5 kmph/sec. (from Table 4.7); t = 2 secs.

$$d_1 = 0.28\, V_b\, t = 0.28 \times 80 \times 2 = 44.8 \text{ m}$$

$$d_2 = 0.28\, V_b\, T + 2.s$$

$$s = (0.2\, V_b + 6) = 0.2 \times 80 + 6 = 22 \text{ m}$$

$$T = \sqrt{\frac{14.4\, s}{A}} = \sqrt{\frac{14.4 \times 22}{2.5}} = 11.3 \text{ secs.}$$

$$d_2 = 0.28 \times 80 \times 11.3 + 2 \times 22 = 297 \text{ m}$$

$$d_3 = 0.28\, V\, T = 0.28 \times 96 \times 11.3 = 303.7 \text{ m}$$

O.S.D. on one-way traffic road = $d_1 + d_2 = 341.8$ m; say 342 m

O.S.D. on two-way traffic road = $d_1 + d_2 + d_3 = 645.5$ m; say 646 m

4.3.4 Sight Distance at Intersections

It is important that on all approaches of intersecting roads, there is a clear view across the corners from a sufficient distance so as to avoid collision of vehicles. This is all the more important at uncontrolled intersections. The sight line is obstructed by structures or other objects at the corners of the intersections. The area of unobstructed sight formed by the lines of vision is called the sight triangle. See Fig. 4.17.

The design of sight distance at intersections may be based on three possible conditions :

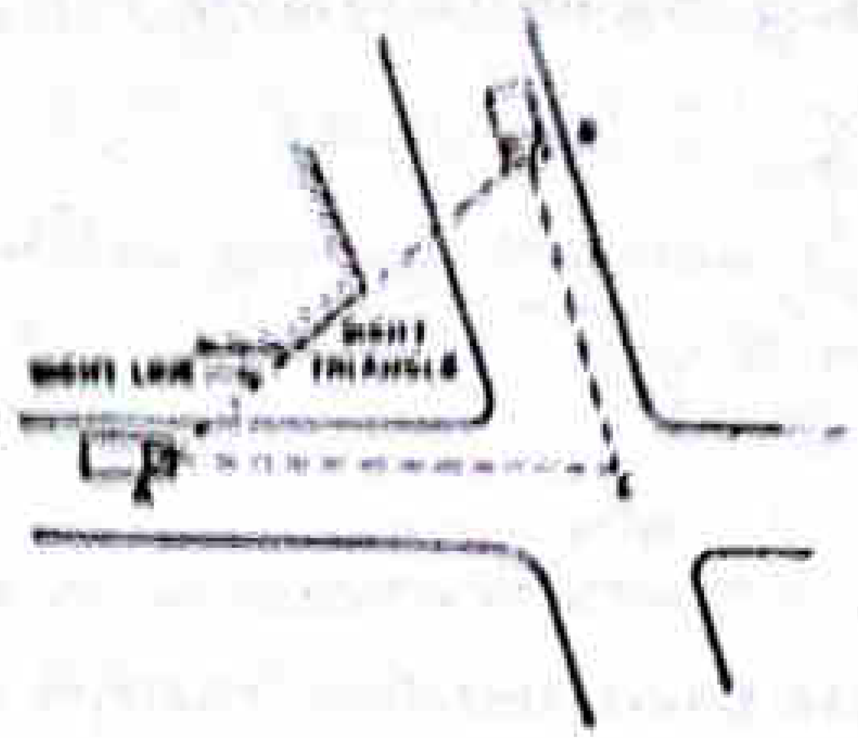


Fig. 4.17 Sight Distance at Intersection

(i) *Enabling the approaching vehicle to change speed* : The sight distance should be sufficient to enable either one or both the approaching vehicles to change speed to avoid collision. The vehicle approaching from the minor road should slow down. The total reaction time required for the driver to decide to change speed may be assumed as two seconds and at least one more second will be needed for making the change in speed. Hence the two sides AC and BC of the sight triangle along the intersection approaches upto the conflict point C should be atleast equal to the distance covered by a vehicle traveling at design speed in two seconds. But this sight distance being too less, should be increased in all possible cases.

(ii) *Enabling approaching vehicle to stop* : In this case, the distances for the approaching vehicle should be sufficient to bring either one or both of the vehicle to a stop before reaching a point of collision. Hence the two sides AC and BC of the sight triangle should each be equal to the safe stopping distance. In almost all uncontrolled intersections one of the two cross roads is a preference highway or a through road or a major road. Thus it is the responsibility of the drivers on the minor road who would cross or enter this main road, to stop or change speed to avoid collision. The traffic of the minor road is generally controlled by an appropriate traffic sign. In such a case the sight distance for a minor road should be atleast equal to the SSD for the design speed of that road. The sight distance requirement of stopping is higher than that of condition (i) above and hence is safe as vehicles can stop if necessary.

(iii) *Enabling stopped vehicle to cross a main road* : This case is applicable when the vehicles entering the intersection from the minor road are controlled by stop sign and so these vehicles have to stop and then proceed to cross the main road. In such a situation, the sight distance available from the stopped position of the minor road should be sufficient to enable the stopped vehicle to start, accelerate and cross the main road, before another vehicle travelling at its design speed on the main road reaches the intersection. The time T required for the stopped vehicle to cross the main road would depend upon (a) reaction time of the driver (b) width of the main road (c) acceleration, and (d) length of vehicle. Thus the minimum sight distance to fulfil this condition is the distance travelled by a vehicle on the main road at design speed during this time 'T'.

From safety considerations, the sight distance at *uncontrolled intersections* should *therefore* fulfil all the above three conditions. The higher of the three values may be taken at unsignalised intersections at grade, except at rotaries. The IRC recommends that

at uncontrolled intersections, sufficient visibility should be provided such that the sight distance of each road is atleast equal to the SSD corresponding to the design speed of the road. If the sight triangle available is less than the desirable minimum size due to unavoidable reasons, the vehicles approaching the intersection may be warned or controlled by suitable signs.

At rotaries the sight distance should be at least equal to the safe stopping distance for the design speed of the rotary. At signalized intersections, the above three requirements are not applicable.

At *priority intersections* where a minor road crosses a major road, the traffic on the minor road may be controlled by stop or give-way sign to give priority to the traffic on the major road. The visibility distance available along the minor road should be sufficient to enable the drivers stop their vehicles. The visibility distance along the major road depends upon the time required for the stopped vehicles approaching from the minor road to evaluate the gaps between the vehicles on the major road, to accelerate and to cross the major road safety. IRC recommends that a minimum visibility distance of 15 m along the minor road and a distance of 220, 180, 145 and 110m along the major and corresponding to the design speeds of 100, 80, 65 and 50 kmph respectively may be provided.

4.4 DESIGN OF HORIZONTAL ALIGNMENT

4.4.1 General

Often changes in the direction are necessitated in highway alignment due to obligatory points as discussed in Chapter 3. Various design factors to be considered in the horizontal alignment are design speed, radius of circular curves, type and length of transition curves, superelevation and widening of pavement on curves.

The alignment should enable consistent, safe and smooth movement of vehicles operating at design speeds. It is hence necessary to avoid those sharp curves and reverse curves which could not be conveniently negotiated by the vehicles at design speed. Improper design of horizontal alignment of roads would necessitate speed changes resulting in increased vehicle operation cost and higher accident rate.

4.4.2 Design Speed

The overall design of geometrics of any highway is a function of the design speed.

The design speed is the main factor on which geometric design elements depends. The sight distances, radius of horizontal curve, superelevation, extra widening of pavement, length of horizontal transition curve and the length of summit and valley curve are all dependent on design speed.

The design speed of roads depends upon (i) class of the road and (ii) terrain. The speed standards of a particular class of road thus depends on the classification of the terrain through which it passes. The terrains have been classified as plain, rolling, mountainous and steep, depending on the cross slope of the country as given below :

Terrain classification	Cross slope of the country, percent
Plain	0-10
Rolling	10-25
Mountainous	25-60
Steep	greater than 60

The design speed (ruling and minimum) standardized by the IRC for different classes of roads on different terrains in rural areas are given in Table 4.8. The ruling design speeds are the guiding criteria for the geometric design. However, minimum design speeds may be accepted where site conditions or economic considerations warrant. The ruling design speeds suggested for the National and State Highways of our country passing through plain terrain is 100 kmph and through rolling terrain is 80 kmph.

Table 4.8 Design Speeds on Rural Highways

Road classification	Design speed in kmph for various terrains							
	Plain		Rolling		Mountainous		Steep	
	Ruling	Min.	Ruling	Min.	Ruling	Min.	Ruling	Min.
National & 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

Speed restrictions have been imposed for heavy vehicles (other than passenger cars) like buses, trucks and vehicles pulling trailer units under *Motor Vehicles Act*. Also speed limits are specified for different categories of vehicles by regulatory signs on urban roads and on some stretches of rural highway when warranted due to safety considerations.

The recommended design speeds for different classes of urban roads are :

(i) for arterial roads 80 kmph,

(ii) sub-arterial roads 60 kmph,

(iii) collector streets 50 kmph and

(iv) local streets 30 kmph

4.4.3 Horizontal Curves

A horizontal highway curve is a curve in plan to provide change in direction to the central line of a road. When a vehicle traverses a horizontal curve, the centrifugal force acts horizontally outwards through the centre of gravity of the vehicle.

The centrifugal force developed depends on the radius of the horizontal curves and the speed of the vehicle negotiating the curve. This centrifugal force is counteracted by the transverse frictional resistance developed between the tyres and the pavement which enables the vehicle to change the direction along the curve and to maintain the stability of the vehicle. Centrifugal force P is given by the equation :

$$P = \frac{W v^2}{gR}$$

Here P = centrifugal force, kg

W = weight of the vehicle, kg

R = radius of the circular curve, m

v = speed of vehicle, m/sec

g = acceleration due to gravity = 9.8 m/sec^2

The ratio of the centrifugal force to the weight of the vehicle, P/W is known as the *centrifugal ratio* or the *impact factor*. The centrifugal ratio is thus equal to v^2/gR.

The centrifugal force acting on a vehicle negotiating a horizontal curve has two effects :

(i) *Tendency to overturn the vehicle outwards about the outer wheels and*

(ii) *Tendency to skid the vehicle laterally, outwards.*

The analysis of stability of those two conditions against overturning and transverse skidding of the vehicles negotiating horizontal curves without superelevation are given below :

(i) Overturning effect

The centrifugal force that tends the vehicle to overturn about the outer wheels B on horizontal curve without superelevation is illustrated in Fig. 4.18. The overturning moment due to centrifugal force P is P × h; this is resisted by the restoring moment due to weight of the vehicle W and is equal to W.b/2, where h is the height of the center of gravity of the vehicle above the road surface and b is the width of the wheel base or the wheel track of the vehicle.

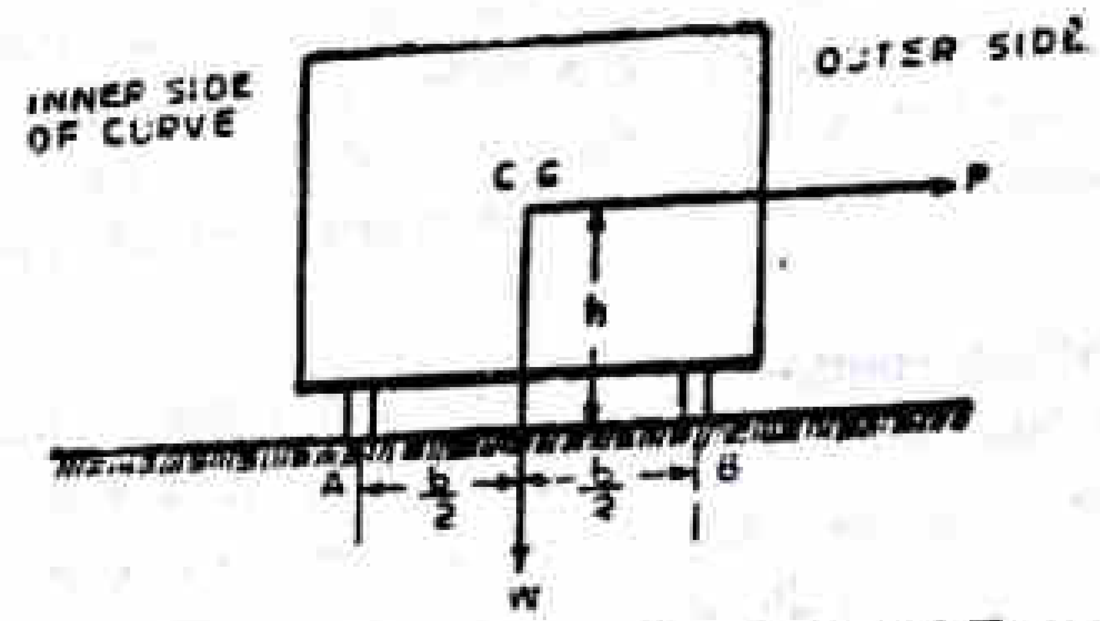


Fig. 4.18 Overturning due to Centrifugal Force

The equilibrium condition for overturning will occur when Ph = Wb/2, or when P/W = b/2h. This means that there is danger of overturning when the centrifugal ratio P/W or v^2/gR attains a values of b/2h.

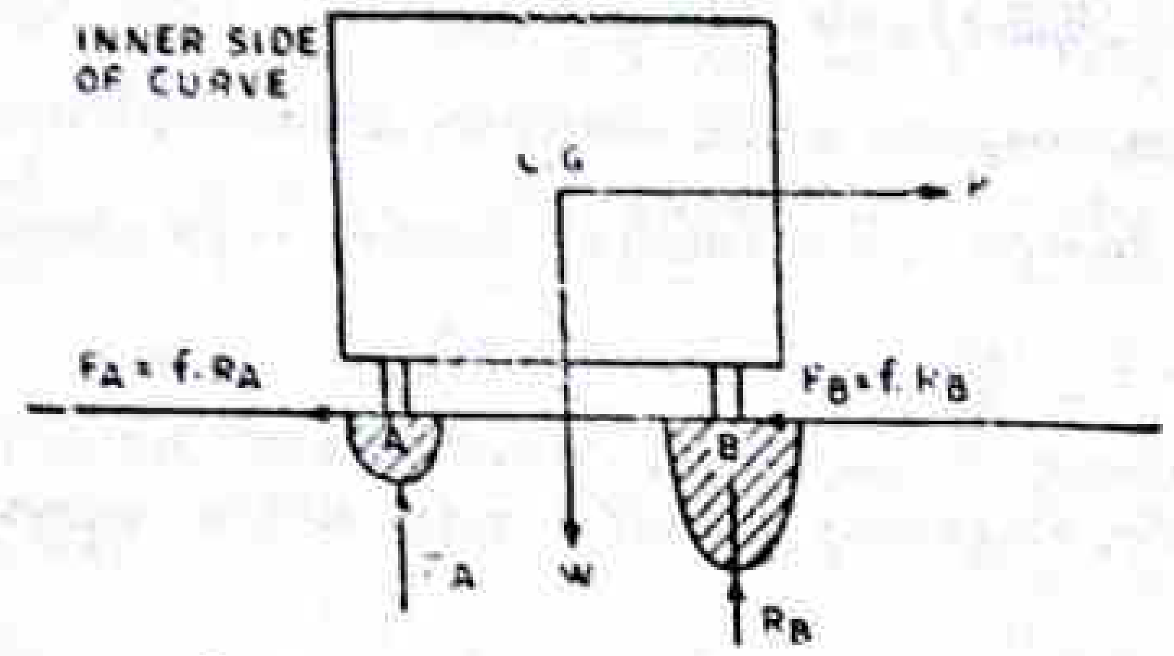
(ii) Transverse skidding effect

The centrifugal force developed has also the tendency to push the vehicle outwards in the transverse direction. If the centrifugal force P developed exceeds the maximum possible transverse skid resistance due to the friction, the vehicle will start skidding in the transverse direction. Refer Fig. 4.19. The equilibrium condition for the transverse skid resistance developed is given by :

$$P = F_A + F_B = f(R_A + R_B) = fW$$

In the above relation, f is the coefficient of friction between the tyre and the pavement surface in the transverse direction, R_A and R_B are normal reactions at the wheels A and B such that $(R_A + R_B)$ is equal to the weight W of the vehicle, as no superelevation has been provided in this case.

Since P = f W, the centrifugal ratio P/W is equal to 'f'. In other words when the centrifugal ratio attains a value equal to the coefficient of lateral friction there is a danger of lateral skidding.



SHADED AREAS SHOW THE PRESSURE UNDER THE INNER AND OUTER WHEELS A AND B

Fig. 4.19 Skidding Effect due to Centrifugal Force

Thus to avoid overturning and lateral skidding on a horizontal curve, the centrifugal ratio should always be less than b/2h and also 'f'.

The vehicle negotiating a horizontal curve with no superelevation has to fully depends on the coefficient of friction 'f' to resist the lateral skidding. The centrifugal force may be enough to cause overturning or lateral skidding of the vehicle if either the speed of the vehicle is high or the radius of the curve is less. In such a case the vehicle would skid and not overturn if the value of 'f' is less than b/2h. On the other hand the vehicle would overturn on the outer side before skidding if the value of b/2h is lower than 'f'. Thus the relative danger of lateral skidding and overturning depends on whether f is lower or higher than b/2h.

If the pavement is kept horizontal across the alignment, the pressure on the outer wheels will be higher due to the centrifugal force acting outwards and hence the reaction R_B at the outer wheel would be higher. The difference in pressure distribution at inner and outer wheels has been indicated in Fig. 4.19. When the limiting equilibrium condition for overturning occurs the pressure at the inner wheels becomes equal to zero.

4.4.4 Superelevation

In order to counteract the effect of centrifugal force and to reduce the tendency of the vehicle to overturn or skid, the outer edge of the pavement is raised with respect to the inner edge, thus providing a transverse slope throughout the length of the horizontal curve. This transverse inclination to the pavement surface is known as superelevation or cant or banking. The superelevation 'e' is expressed as the ratio of the height of outer edge with respect to the horizontal width. From Fig. 4.20 it may be seen that superelevation,

$$e = \frac{NL}{ML} = \tan\theta$$

In practice the inclination θ with the horizontal is very small and the value of tan θ seldom exceeds 0.07. Therefore the value of than θ is practically equal to sin θ.

Hence, $e = \tan\theta \approx \sin\theta = \frac{E}{B}$ which is measured as the ratio of the relative elevation of the outer edge, E to width of pavement, B. This is more convenient to measure.

If e is the superelevation rate and E is the total superelevated height of outer edge, the total rise in outer edge of the pavement with respect to the inner edge = NL = E = eB.

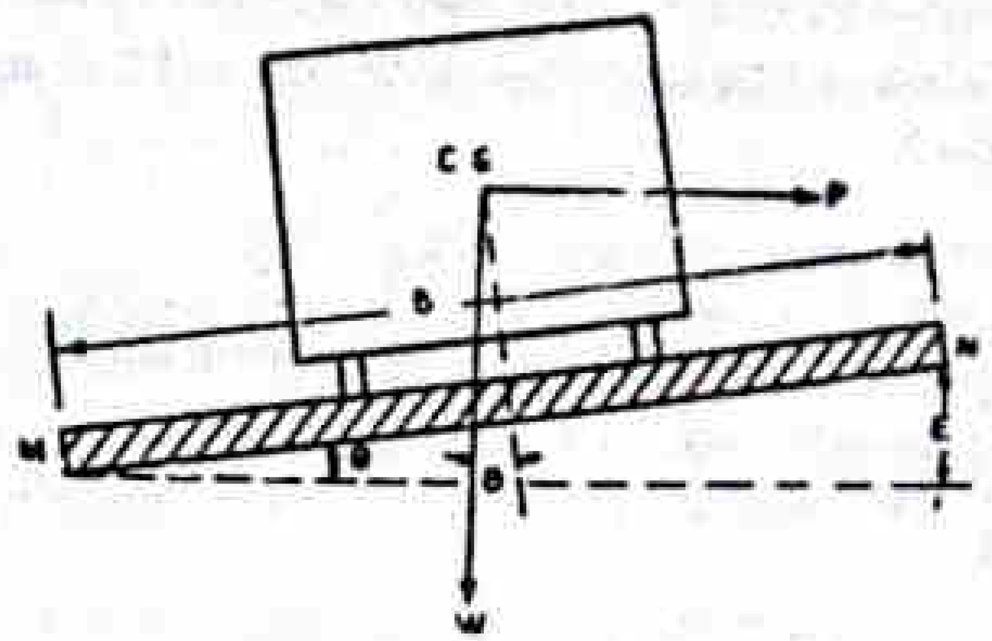


Fig. 4.20 Superelevated Pavement Section

Analysis of superelevation

The forces acting on the vehicle while moving on a circular curve of radius R metres at speed of v m/sec are

(i) the centrifugal force $P = Wv^2/gR$ acting horizontally outwards through the center of gravity, CG

(ii) the weight W of the vehicle acting vertically downwards through the CG

(iii) the frictional force developed between the wheels and the pavement counteractions transversely along the pavement surface towards the center of the curve.

The centrifugal force is thus opposed by corresponding value of the friction developed and by a component of the force of gravity due to the superelevation provided. Figure 4.21 shows the cross section of a pavement with all the forces acting on the vehicle resolved parallel and perpendicular to the inclined road surface. Considering the equilibrium of the components of forces acting parallel to the plane, (P cos θ) the component of centrifugal force is opposed by (W sin θ) the component of gravity and the frictional forces F_A and F_B.

For equilibrium condition,

$$P\cos\theta = W\sin\theta + F_A + F_B$$

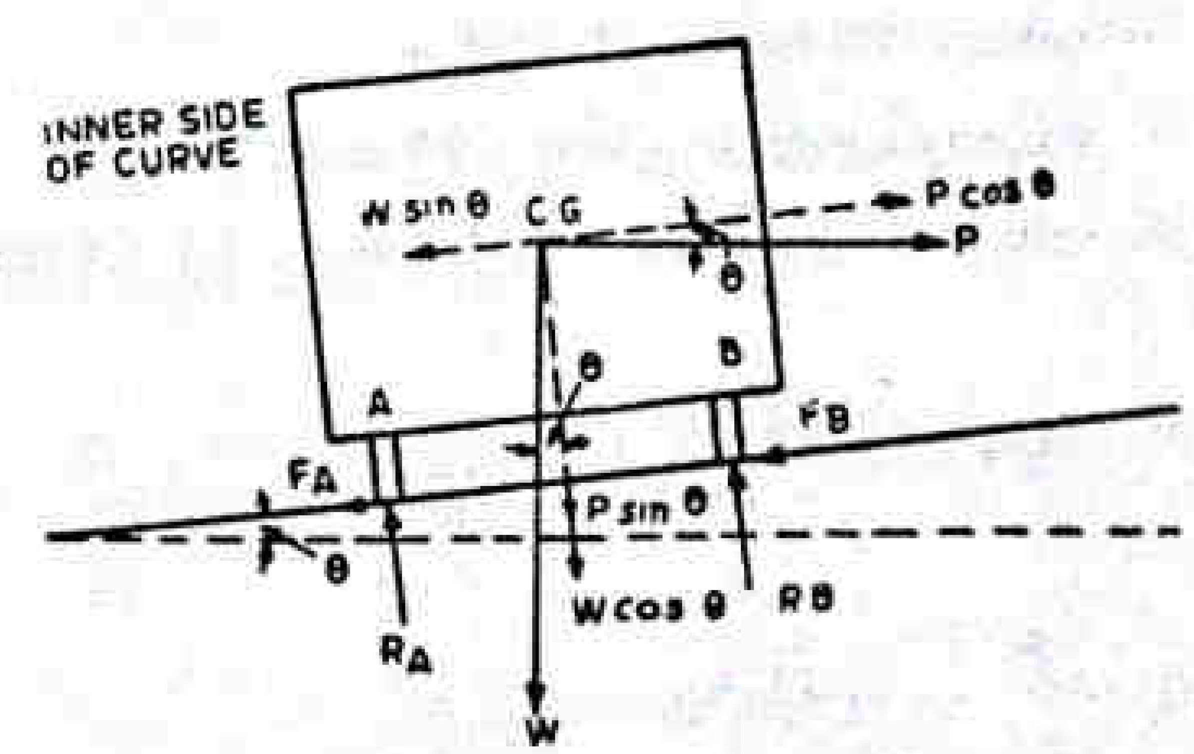


Fig. 4.21 Analysis of Superelevation

The limiting equilibrium is reached when the full values of the frictional forces are developed and the values of F_A and F_B reach their maximum value of $f \times R_B$ and $f \times R_A$ respectively where 'f' is the coefficient of lateral friction and R_A and R_B are the normal reactions at wheels A and B.

Therefore,
$$P \cos\theta = W \sin\theta + f(R_A + R_B)$$
$$= W \sin\theta + f(W \cos\theta + P \sin\theta)$$

i.e.,
$$P(\cos\theta - f\sin\theta) = W \sin\theta + f W \cos\theta$$

Dividing by $W \cos\theta$,

$$\frac{P}{W}(1 - f\tan\theta) = \tan\theta + f$$

$$\frac{P}{W} = \frac{\tan\theta + f}{1 - f\tan\theta}$$

The value of coefficient of lateral friction, 'f' is taken as 0.15 for design purposes, (See article 4.1.1). The value of $\tan\theta$ or transverse slope due to superelevation seldom exceeds 0.07 or about 1/15. Hence the value of $f \tan\theta$ is about 0.01. Thus the value of $(1 - f\tan\theta)$ in the above equation is equal to 0.99 and may be approximated to 1.0.

Therefore,
$$\frac{P}{W} \approx \tan\theta + f = e + f$$

But
$$\frac{P}{W} = \frac{v^2}{gR}$$

Therefore,
$$e + f = \frac{v^2}{gR} \qquad (4.7)$$

Here
- e = rate of superelevation = $\tan\theta$
- f = design value of lateral friction coefficient = 0.15
- v = speed of the vehicle, m/sec
- R = radius of the horizontal curve, m
- g = acceleration due to gravity = 9.8 m/sec^2

If the speed of the vehicle is represented as V kmph, the Eq. 4.8 may be written as follows ;

$$E + f = \frac{(0.278V)^2}{9.8R} = \frac{V^2}{127R}$$

i.e.
$$e + f = \frac{V^2}{127R} \qquad (4.8)$$

- V = speed, kmph
- R = radius, surfaces

If the coefficient of friction is neglected or assumed equal to zero, i.e. if f = 0, the *equilibrium superelevation* required to counteract the centrifugal force fully will be given by :

$$e = \frac{v^2}{gR}\ \frac{V^2}{127R}$$

If superelevation is provided according to this formula, the pressures on the outer and inner wheels will be equal; but this will result in a very high value of superelevation. As considerable role is played by the lateral frictional resistance in counteracting the centrifugal force, it is always taken into account. In places where superelevation is not provided due to practical difficulties, i.e. where e = 0 and $f = \frac{v^2}{gR} = \frac{V^2}{127R}$, and the frictional force has to fully counteract the centrifugal ratio. In some types of intersections it is not possible to provide superelevation and in such cases the friction counteracts the centrifugal force fully; with no superelevation, the allowable speed of vehicle negotiating, a turn should be restricted to the condition,

$$f = \frac{v^2}{gR} = \frac{V^2}{127R}, \text{ or } V = \sqrt{127fR}$$

It is possible that at some intersections, a negative superelevation is unavoidable.

Thus the superelevation 'e' required on a horizontal curve depends on the radius of the curve R, speed of the vehicle V and the coefficient of lateral friction or the transverse skid resistance f. Therefore, in order to assess the superelevation e required, the speed is taken as equal to the design speed of the road and the minimum value of transverse skid resistance f for design purpose is standardised equal to 0.15.

Example 4.8

The radius of a horizontal circular curve is 100 m. The design speed is 50 kmph and the design coefficient of lateral friction is 0.15.

(a) calculate the superelevation required if full lateral friction is assumed to develop.

(b) calculate the coefficient of friction needed if no superelevation is provided.

(c) calculate the equilibrium superelevation if the pressure on inner and outer wheels should be equal.

Solution

(a) Superelevation is given by the relation

$$e + f = \frac{v^2}{gR} = \frac{V^2}{127R} \quad \text{(Equation 4.7 \& 4.8)}$$

Here
$$f = 0.15;\ V = 50 \text{ kmph or } v = \frac{50}{3.6} \text{ m/sec.}$$
$$R = 100 \text{ m}$$

$$e + 0.15 = \frac{50^2}{127 \times 100} = 0.917$$

$$e = 0.197 - 0.15 = 0.047$$

i.e., superelevation rate is 1 in 21.2

(b) If no superelevation is provided, e = 0 and friction factor developed,

$$f = \frac{V^2}{127R} = \frac{50^2}{127 \times 100} = 0.917$$

(c) For the pressure on inner and outer wheels to be equal or for equilibrium superelevation counteracting centrifugal force fully, f = 0 and

$$e = \frac{V^2}{127R} = \frac{50^2}{127 \times 100} = 0.917$$

i.e., equilibrium superelevation rate is 1 in 5.1. However this rate of superelevation being very high, cannot be provided.

Maximum superelevation

As per Equation 4.7 and 4.8, the value of superelevation needed increases with increase in speed and with decrease in radius of the curve, for a constant value of coefficient of lateral friction 'f'. From the practical view point it will be necessary to limit the maximum allowable superelevation to avoid very high values of 'e'. This is particularly necessary when the road has to cater for mixed traffic, consisting of fast and slow traffic.

In the case of heavily loaded bullock carts and trucks carrying less dense materials like straw or cotton, the centre of gravity of the loaded vehicle will be relatively high and it will not be safe for such vehicles to move on a road with a high rate of superelevation. Because of the slow speed, the centrifugal force will be negligibly small in the case of bullock carts. Hence to avoid the danger of toppling of such loaded slow moving vehicles, it is essential to limit the value of maximum allowable superelevation. Indian Roads Congress had fixed the maximum limit of superelevation in plain and rolling terrains and is snow bound areas as 7.0 percent taking such mixed traffic into consideration. However, on hill roads not bound by snow a maximum superelevation upto 10 percent has been recommended. On urban road stretches with frequent intersections, it may be necessary to limit the maximum superelevations to 4.0 percent, keeping in view the convenience in construction and that of turning movements of vehicles.

Minimum Superelevation

From drainage considerations it is necessary to have a minimum cross slope to drain off the surface water. If the calculated superelevation from Equation 4.8 worksout to be equal to or less than the camber of the road surface, then the minimum superelevation to be provided on horizontal curve may be limited to the camber of the surface. Thus after the elimination of the crown a uniform cross slope equal to the camber is maintained from outer to inner edge of pavement at the circular curve. In very flat curves with large radius

the centrifugal force developed will be very small and in such cases the normal camber may be retained on the curves. Though this practice will cause a negative superelevation on the outer half of the pavement due to the normal camber, the centrifugal force together with this negative superelevation would be considerably less than the allowable friction coefficient on such curves. The IRC recommendation giving the radii of horizontal curves beyond which normal cambered section may be maintained and no superelevation is required for curves, are presented in Table 4.9, for various design speeds and rates of cross slope.

Table 4.9 Radii beyond which Superelevation is not required

Design speed (kmph)	Radius (metre) of horizontal curve for camber of :				
	4 %	3 %	2.5 %	2 %	1.7 %
20	50	60	70	90	100
25	70	90	110	140	150
30	100	130	160	200	240
35	140	180	220	270	320
40	180	240	280	350	420
50	280	370	450	550	650
60	470	620	750	950	1100
80	700	950	1100	1400	1700
100	1100	1500	1800	2200	2600

Superelevation Design

Design of superelevation for mixed traffic conditions is complex problem, as different vehicles ply on the road with a wide range of speeds. To superelevate the pavement upto the maximum limit so as to counteract the centrifugal force fully, neglecting the lateral friction is safer for fact moving vehicles. But for slow moving vehicles this may quite inconvenient. On the contrary to provide lower value of superelevation thus relying more on the lateral friction would be unsafe for fast moving vehicles. As a compromise and from practical considerations it is suggested that the superelevation should be provided to fully counteract the centrifugal force due to 75 percent of the design speed, (by neglecting lateral friction developed) and limiting the maximum superelevation to 0.07 (except on hill roads, not bound by snow where the maximum allowable value is 0.1).

Steps for superelevation design :

Various steps in the design of superelevation in practice may be summarized as given below :

Step (i) The superelevation for 75 percent of design speed (v m/sec or V kmph) is calculated neglecting the friction

$$e = \frac{(0.75\,v)^2}{gR} \text{ or } \frac{(0.75\,V)^2}{127\,R}$$

i.e.,

$$e \approx \frac{V^2}{225\,R} \tag{4.9}$$

Step (ii) If the calculated value of 'e' is less than 7% or 0.07 the value so obtained is provided. If the value of 'e' as per equation 4.9 exceeds 0.07 then provide the maximum superelevation equal to 0.07 and proceed with steps (iii) or (iv).

Step (iii) Check the coefficient of friction developed for the maximum value of e = 0.07 at the full value of design speed,

$$F = \left(\frac{v^2}{gR} - 0.07\right) \quad (4.10)$$

$$= \left(\frac{V^2}{127\,R} - 0.07\right)$$

If the value of f thus calculated is less than 0.15, the superelevation of 0.07 is safe for the design speed. If not, calculate the restricted speed as given in step (iv).

Step (iv) As an alternative to step (iii), the allowable speed (v_a m/sec. or V_a kmph) at the curve is calculated by considering the design coefficient of lateral friction and the maximum superelevation, i.e.,

$$e + f = 0.07 + 0.15$$

$$= 0.22 = \frac{v_a^{\,2}}{gR} = \frac{V_a^{\,2}}{127\,R}$$

calculate the safe allowable speed,

$$v_a = \sqrt{0.22\,gR} = \sqrt{2.156\,R} \text{ m/sec}$$

or

$$V_a = \sqrt{27.94\,R} \text{ kmph} \quad (4.11)$$

If the allowable speed, as calculated above is higher than the design speed, then the design is adequate and provides a superelevation of 'e' equal to 0.07. If the allowable speed is less than the design speed, the speed is limited to the allowable speed V_a kmph calculated above.

Appropriate warning sign and speed limit regulation sign are installed to restrict and regulate the speed at such curves when the safe speed V_a is less than the design speed V. For important highways, it is desirable to design the road without speed restriction at curves, as far as possible. Hence if site conditions permit, the curve should be re-aligned with a larger radius of curvature so that the design speed could be maintained (See Art. 4.4.4 and Table 4.10 for radius of horizontal curve).

Example 4.9

A two lane road with design speed 80 kmph has horizontal curve of radius 480 m. Design the rate of superelevation for mixed traffic. By how much should the outer edges of the pavement be raised with respect to the centre line, if the pavement is rotated with respect to the centre line and the width of the pavement at the horizontal curve is 7.5 m.

Solution

For mixed traffic conditions the superelevation should fully counteract the centrifugal force for 75% of design speed.

Hence using Equation 4.9.

$$e = \frac{V^2}{225\,R} = \frac{80^2}{225 \times 480} = 0.059$$

Since this value is less than 0.07, the superelevation of 0.059 may be adopted.

The total width of pavement B = 7.5 m.

Raising of outer edge with respect to centre

$$= E = \frac{B.e}{2} = \frac{7.5}{2} \times 0.059 = 0.22 \text{ m}$$

Example 4.10

Design the rate of superelevation for a horizontal highway curve of radius 500 m and speed 100 kmph.

Solution

For mixed traffic conditions, superelevation is given by Eq. 4.9.

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

$$V = 100 \text{ kmph}$$

$$R = 500 \text{ m}$$

$$e = \frac{100^2}{225 \times 500} = 0.089$$

As the value is greater than the maximum superelevation of 0.07, the actual superelevation to be provided is restricted to 0.07.

Check for coefficient of lateral friction developed for full speed using Eq. 4.10.

$$f = \frac{V^2}{127\,R} - 0.07 = \frac{100^2}{127 \times 500} - 0.07$$

$$= 0.157 - 0.07 = 0.087$$

As the value is less than 0.15, the design is safe with a superelevation of 0.07.

Example 4.11

The design speed of a highway is 80 kmph. There is a horizontal curve of radius 200 m on a certain locality. Calculate the superelevation needed to maintain this speed. If the maximum superelevation of 0.07 is not to be exceeded, calculate the maximum allowable speed on this horizontal curve as it is not possible to increase the radius. Safe limit of transverse coefficient of friction is 0.15.

Solution

The problem may be solved by considering 75 percent design speed for finding the superelevation or counteract the centrifugal force fully using Eq. 4.9.

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

i.e.,

$$e = \frac{80^2}{225 \times 200} = 0.142$$

Maximum allowable value of e is to be limited to 0.07.

Check for the value of friction developed,

$$f = \frac{V^2}{127\,R} - 0.07$$

$$= \frac{80^2}{127 \times 200} - 0.07 = 0.18$$

As this value is greater than the maximum allowable safe friction coefficient of 0.15 and also as the radius can not be increased, the speed has to be restricted.

Hence the maximum allowable speed (V_a kmph) on this curve is obtained by assuming the full value of design friction coefficient on 0.15. This is given by the Eq. 4.11.

$$V_a = \sqrt{27.94R} = 74.75 \text{ kmph}$$

Hence the speed may be restricted to less than 74 or say 70 kmph at this curve.

Example 4.12

A major District Road with thin bituminous pavement surface in low rainfall area has horizontal curve of radius 1400 m. If the design speed is 65 kmph, what should be the superelevation ? Discuss.

Solution

Using Eq. 4.9,

$$e = \frac{V^2}{225\,R} = \frac{65^2}{225 \times 1400} = 0.0134$$

The superelevation value required is only 0.0134 which is even less than the normal cross slope required to drain off the surface water. The recommended camber for thin bituminous pavement in low rainfall area (Table 4.1) is 2% or 0.02. The radius beyond which no superelevation is required for a speed of 65 kmph and 2% camber is 950 m as per the IRC (See table 4.9). As the radius of the horizontal curve in this case is 1400 m, there is no necessity of providing superelevation; therefore the normal camber of 2% may be retained at the horizontal curve.

However, check for safety against centrifugal force at design speed along with the negative superelevation at the outer half of the pavement due to the normal camber.

$$\text{Net transverse skid resistance} = -e + f = -0.02 + 0.15 = 0.13$$

$$\text{Centrifugal ratio} = \frac{V^2}{127\,R} = \frac{65^2}{127 \times 1400} = 0.024$$

As this value of 0.024 is considerably lower than the net transverse skid resistance of 0.13 available at the curve, this horizontal curve with normal cambered section is quite safe for a design speed of 65 kmph.

Attainment of Superelevation

Introducing superelevation on a horizontal curve in the field is an important feature in construction. The road cross section at the straight portion is cambered with the crown at the centre of the pavement and sloping down towards the edges. But the cross section in the circular curve portion of the road is superelevated with a uniform tilt sloping down from the outer edge of the pavement up to inner edge. These may be seen from *sections* at A and E of Fig. 4.24. Thus the crowned camber sections at the straight before the start of the transition curve should be changed to a single cross slope equal to the desired superelevation at the beginning of the circular curve. This change may be conveniently attained at a gradual and uniform rate throughout the transition length of the horizontal curve. The full superelevation is attained by the end of transition curve or at the beginning of the circular curve.

The attainment of superelevation may be split up into two parts :

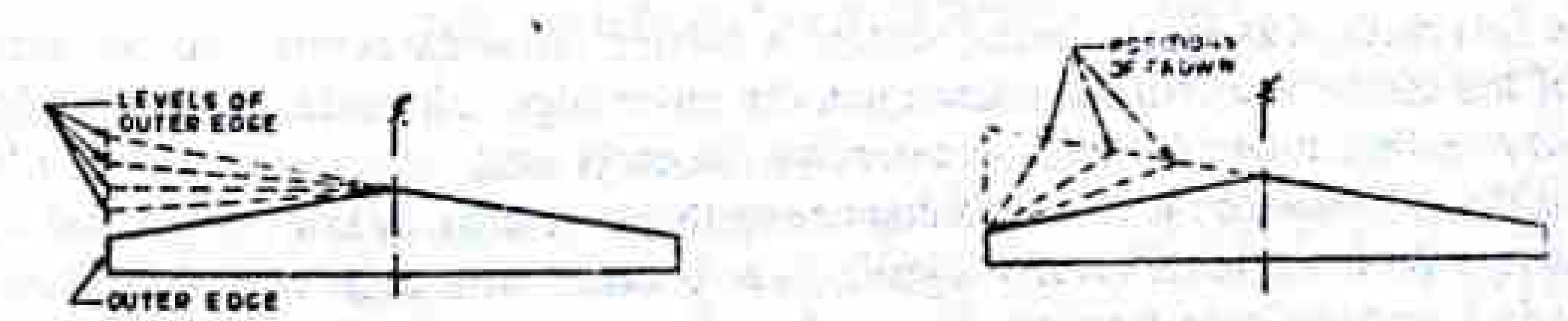
(a) Elimination of crown of the cambered section

(b) Rotation of pavement to attain full superelevation

Elimination of crown of the cambered section

This may be done by two methods. In the first method, the outer half of the cross slope is rotated about the crown at a desired rate such that the surface falls on the same plane as the inner half and the elevation of the centre line is not altered. (Ref. Fig. 4.22a).

The outer half of the cross slope is brought to level or horizontal (by rotating about the crown line) at the start of the transition curve or at tangent point T.P. See cross section at B in Fig. 4.24. Subsequently the outer half is further rotated so as to obtain uniform cross slope equal to the camber, as shown in Fig. 4.22 (a) and in cross section C of Fig. 4.24.



(a) Outer edge rotated about the crown *(b) Crown shifted outwards (diagonal crown method)*

Fig. 4.22 Elimination of Crown of Cambered Section

Thus no point on the curve will have a negative superelevation at the outer half of the pavement event at the start of the transition curve. This method has a drawback that the surface drainage will not be proper at the outer half, during a short stretch of the road with a cross slope less than the camber between point A and C in Fig. 4.24.

In the second method of eliminating the crown, known as diagonal crown method, the crown is progressively shifted outwards, thus increasing the width of the inner half of cross section progressively. This method is not usually adopted as a portion of the outer half of the pavement has increasing values of negative superelevation on to a portion of the outer half, before the crown is eliminated (see Fig. 4.22 b).

Rotation of pavement to attain full superelevation

When the crown of the camber is eliminated, the superelevation available at this section is equal to the camber. But the superelevation to be provided at the beginning of circular curve may be greater than the camber in many cases when the design superelevation is more than the minimum. Hence the pavement section will have to be rotated further till the desired banking is obtained.

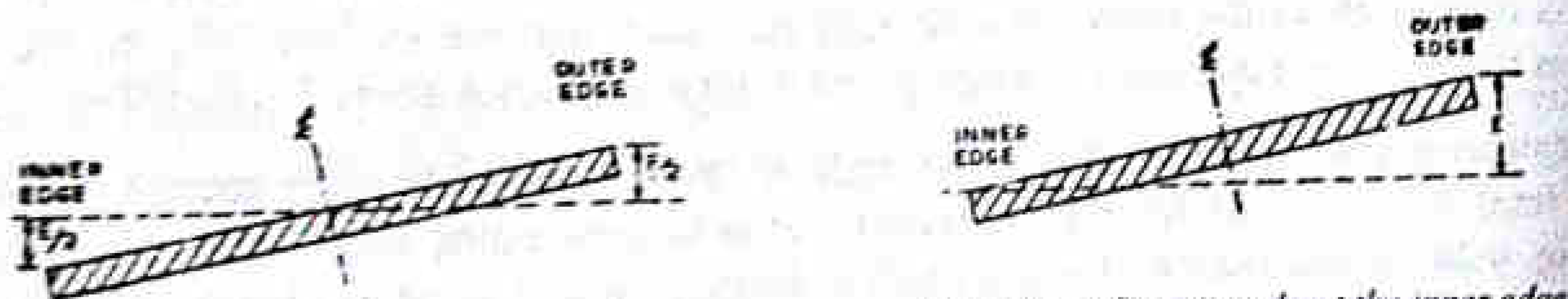
As an example, if the specified camber in a bituminous pavement surface is 0.02 and the design superelevation is 0.07, the camber is first eliminated resulting in a superelevation of 0.02 and then the cross slope is further increased till it attains the full superelevation of 0.07. If the designed superelevation is 'e' and the total width of the pavement at the horizontal curve is 'B', the total banking of the outer edge of the pavement with respect to the inner edge is equal to E = B.e.

There are two methods of rotating the pavement cross section to attain the full superelevation after the elimination of the camber.

(i) By rotating the pavement cross section about the centre line, depressing the inner edge and raising the outer edge each by half the total amount of superelevation, i.e. by E/2 with respect to the centre.

(ii) By rotating the pavement cross section about the inner edge of the pavement section raising both the centre as well as the outer edge of the pavement such that the outer edge is raised by the full amount of superelevation, E with respect to the inner edge.

The two methods are shown in Fig. 4.23.



Method (i) Rotating about centre line *Method (ii) Rotating about the inner edge*

Fig. 4.23 Rotation of Pavement Section to attain Full Superelevation

In the first method as the pavement section is rotated about the centre line, the vertical profile of the centre line remains unchanged; the outer edge is banked and inner edge is depressed resulting in an advantage in balancing the earth work. The disadvantage of this method is the drainage problem due to depressing the inner edge below the general level. The drainage problem is of greater significance in areas with high rain fall when the subgrade is in cutting or in level terrain. If the subgrade is in embankment or when the road has a significant gradient to facilitate longitudinal drainage, there will be no drainage problem.

The second method of rotating about the inner edge is preferably in very flat terrain in high rain fall areas, when the road is not taken on embankment, in order to avoid the drainage problem. But the entire pavement width and outer shoulder should also be raised with respect to the inner edge by additional earth fill. In this case the centre of the pavement is also raised, which may be considered as a disadvantage of the method as the vertical alignment of the road is altered.

The attainment of superelevation has been shown in detail in Fig. 4.24. The plan of the horizontal curve including the straight, transition and circular curves is shown in

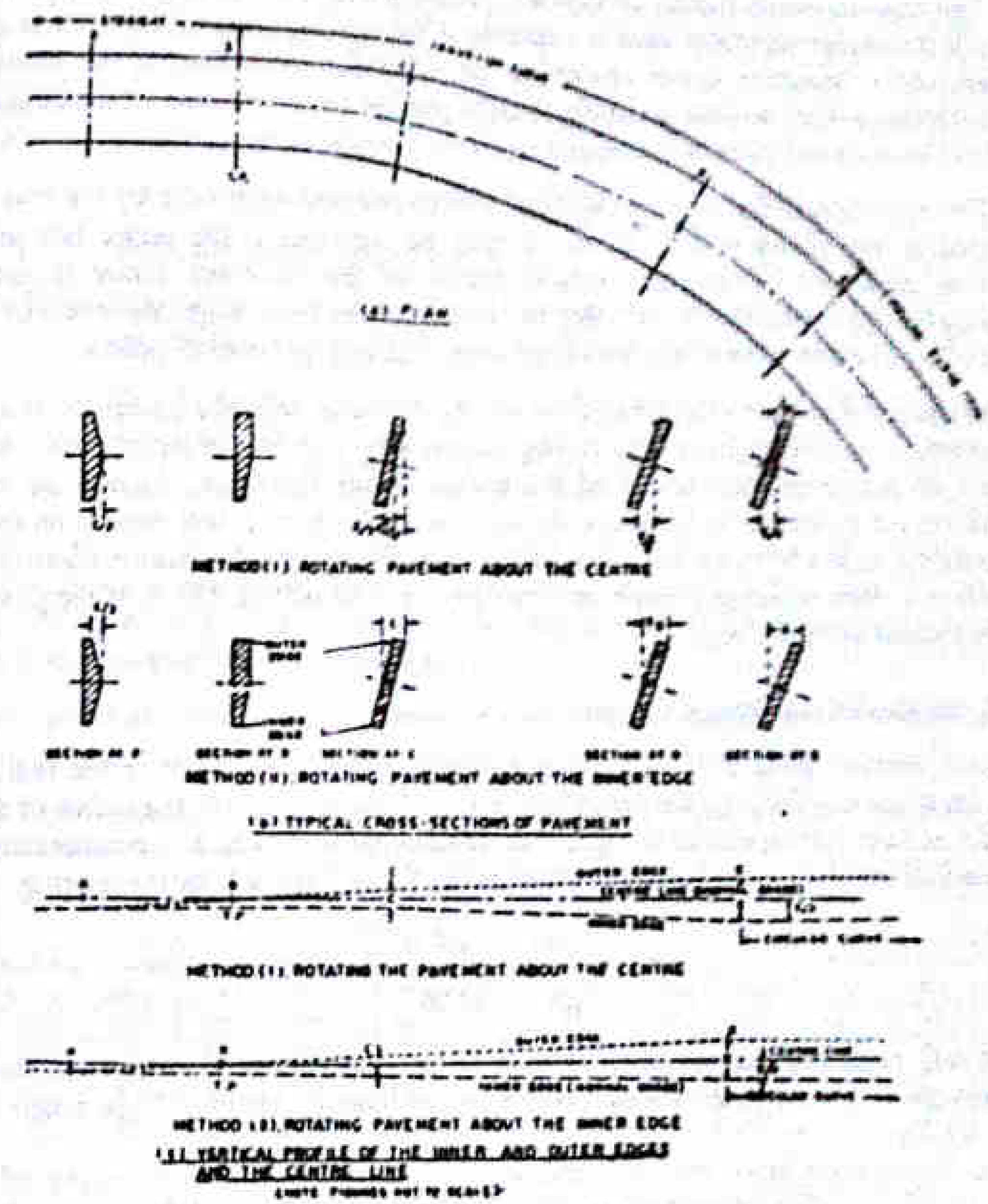


Fig. 4.24 Attainment of Superelevation

Fig. 4.24 a. Elimination of the crown of cambered section, attainment of uniform slope and the two methods of rotating the pavement section to attain full superelevation have been illustrated in Fig. 4.24b. The outer half of the cambered section is raised to a horizontal position between A and B at the same rate of introduction of superelevation along the transition curve of length L_s. Thus at the tangent point B there is no negative superelevation.

When the pavement is rotated about the inner edge, the length AB is given by :

$$\frac{c\,B\,N}{2} = \frac{c\,L_s}{2\,e}$$

where c and e are the rates of camber and superelevation, B is the width of pavement and N is the rate of raising the outer edge of pavement along the transition curve of length L_s. At point C the pavement attains uniform cross slope equal to the camber and the distance BC = AB. The pavement is further rotated at the same rate between C and E to attain full superelevation.

The superelevation should be attained gradually over the full length of transition curve so that the design superelevation is available at the starting point of the circular curve. In cases where transition curve cannot be provided for some reason, two-thirds of the superelevation may be attained at the straight portion before the start of the circular curve and the balance one-third at the beginning of the circular curve.

The vertical profiles of the inner edge, centre line and outer edge by the two methods of rotation are shown in Fig. 4.24c. It may be seen that in the centre line method of rotating pavement section the vertical profile of the pavement centre is not altered throughout the horizontal curve. But by rotating about inner edge, the levels of both the centre line and that outer edges are raised above the original vertical profile.

The superelevation is introduced by raising the outer edge the pavement at a rate not exceeding 1 to 150 in plain and rolling terrain and 1 in 60 on mountainous and steep terrain as per recommendations of the Indian Roads Congress. Hence the length of transition curve needed to introduce the total superelevation E will depend on the rate of introducing superelevation and value of E. Thus the length of transition curve needed to introduce a total superelevation E at a rate of 1 in 150 will be 150 E, if the pavement is rotated about the inner edge.

4.4.5 Radius of Horizontal Curve

For a certain speed of vehicle the centrifugal force is dependent on the radius of the horizontal curve. To keep the centrifugal ratio within a low limit, the radius of the curve should be kept correspondingly high. The centrifugal force which is counteracted by the superelevation and lateral friction is given as per Eq. 4.7 and 4.8, by the relation.

$$e + f = \frac{v^2}{gR} = \frac{V^2}{127R}$$

In this equation, the maximum allowable superelevation rate has been fixed as 7 percent or 0.07 and the design coefficient of lateral friction 'f' is taken as 0.15 (Art. 4.1.2).

Hence, $$e + f = 0.07 + 0.15 = 0.22 = \frac{v^2}{gR} = \frac{V^2}{127R}$$

If the design speed is decided for a highway, then the minimum radius to be adopted can be found from the above relationship.

Thus the ruling minimum radius of the curve for ruling design speed v m/sec. or V/kmph is given by :

$$R_{ruling} = \frac{v^2}{(e+f)\,g} \qquad (4.12)$$

Also, $$R_{ruling} = \frac{V^2}{127\,(e+f)} \qquad (4.13)$$

When the minimum design speed V' kmph is adopted (see Table 4.8) instead of ruling design speed V kmph, the absolute minimum radius of horizontal curve R_{min} is given by :

$$R_{min} = \frac{V'^2}{127\,(e+f)} \qquad (4.14)$$

In the above equations,

v and V = ruling design speeds, in m/sec and kmph respectively

V' = minimum design speed, kmph

E = rate of superelevation; the maximum value of e is taken as 0.07 at all the regions except at hill roads without snow where it is taken as 0.1.

f = design value of transverse skid resistance or coefficient of friction, taken as 0.15

g = acceleration due to gravity = 9.8 m/sec^2

According to the earlier specifications of the IRC, the ruling minimum radius of the horizontal curve was calculated from a speed value, 16 kmph higher than the design speed i.e., (V + 16) kmph. However now the calculations are based on the ruling and minimum design speeds given in Table 4.8.

The ruling and absolute minimum values of radii of horizontal curve of various classes of roads in different terrains (as per the latest IRC specifications) are given in Table 4.10.

Table 4.10 Minimum radii of horizontal curves for different terrain conditions, m

Classification of roads	Plain terrain		Rolling terrain		Mountainous terrain: Area not affected by snow		Mountainous terrain: Snow bound areas		Steep terrain: Area not affected by snow		Steep terrain: Snow bound areas	
	Rulling Mini.	Absolute Mini	Rulling Mini.	Absolute Mini	Rulling Mini.	Absolute Mini	Rulling Mini.	Absolute Mini	Rulling Mini.	Absolute Mini	Rulling Mini	Absolute Mini
NH& SH	360	230	230	155	80	50	90	60	50	30	60	33
MDR	230	155	155	90	50	30	60	33	30	14	33	15
ODR	155	90	90	60	30	20	33	23	20	14	23	15
VR	90	60	60	45	20	14	23	15	20	14	23	15

Note : The values of ruling minimum and absolute minimum radii correspond to the ruling and minimum design speed values given in Table 4.8.

Example 4.13

Calculate the values of ruling minimum and absolute minimum radius of horizontal curve of a National Highway in plain terrain. Assume ruling design speed and minimum design speed values as 100 and 80 kmph respectively.

Solution

Ruling minimum radius is calculated using Eq. 4.12 or 4.13 for ruling design speed of 100 kmph with the maximum values of e = 0.07 and f = 0.15.

$$R_{ruling} = \frac{V^2}{127\,(e+f)} = \frac{100^2}{127\,(0.07+0.15)} = 357.9 \text{ m say } 360$$

The absolute minimum radius is calculated from the minimum design of V" = 80 kmph, using Eq. 4.14.

$$R_{min} = \frac{V'^2}{127\,(e+f)} = \frac{80^2}{127\,(0.07+0.15)} = 229.1 \text{ m say } 230 \text{ m}$$

Therefore provide rulling minimum radius of 360 m and absolute minimum radius of 230 m.

4.4.6 Widening of Pavement on Horizontal Curves

On horizontal curves, especially when they are not of very large radii, it is common to widen the pavement slightly more than the normal width. The object of providing extra widening of pavements on horizontal curves are due to the following reasons :

(a) An automobile has a rigid wheel base and only the front wheels can be turned; when this vehicle takes a turn to negotiate a horizontal curve, the rear wheels do not follow the same path as that of the front wheels. This phenomenon is called *off tracking.* Normally (at low speeds and up to the design speed when no lateral slipping of rear wheels take place) the rear wheels follow the inner path on the curve as compared with those of the corresponding front wheels. This means that if inner front wheel takes a path on the inner edge of a pavement at a horizontal curve, inner rear wheel will be off the pavement on the inner shoulder. The off-tracking depends on the length of the wheel base of the vehicle and the turning angle or the radius of the horizontal curve negotiated. This is illustrated in Fig. 4.25.

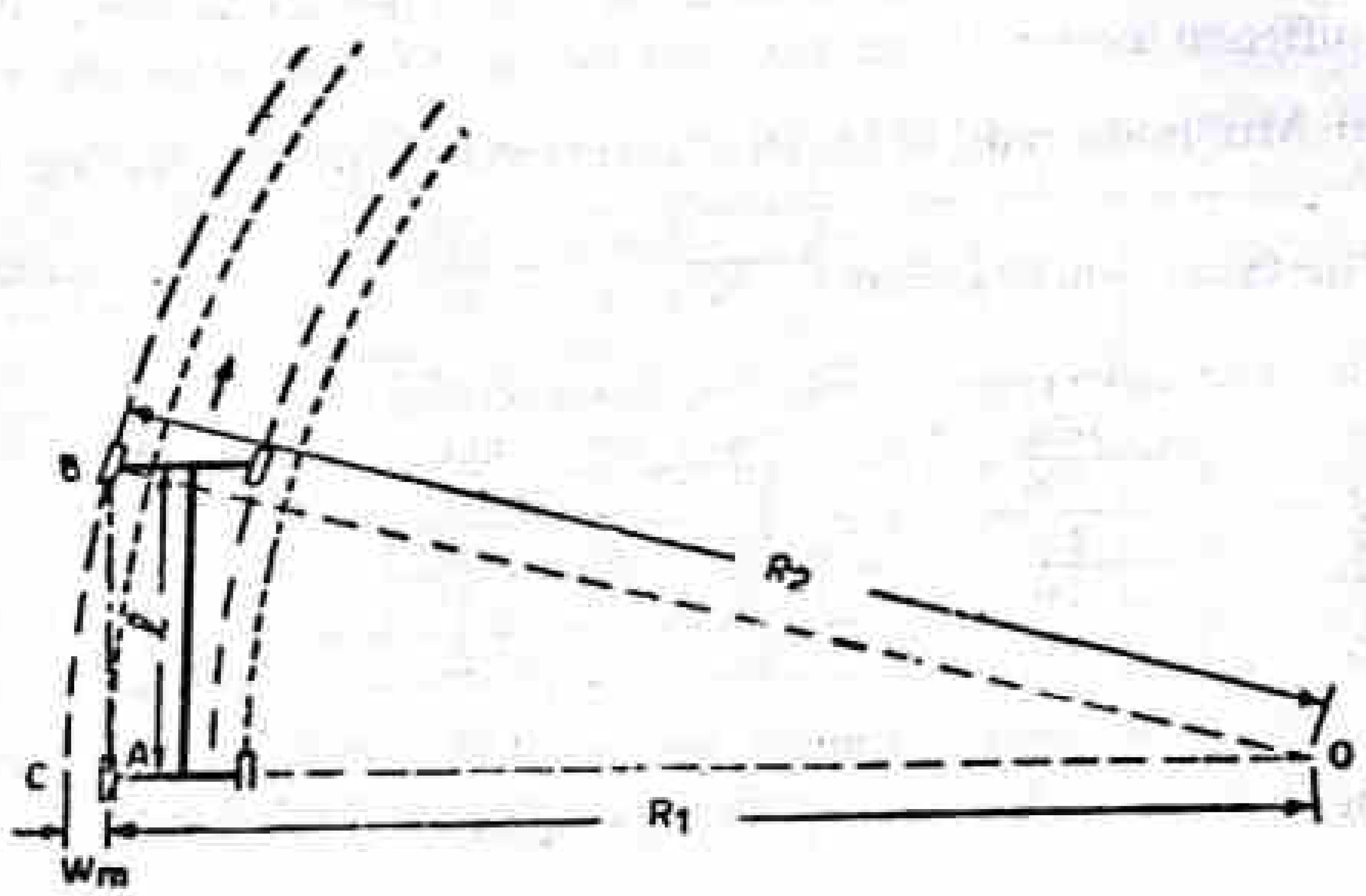


Fig. 4.25 Mechanical Widening on Horizontal Curve

(b) At speeds higher than the design speeds when the superelevation and lateral friction developed are not fully able to counteract the outwards thrust due to the centrifugal force, some transverse skidding may occur and the rear wheels may take paths on the outside of those traced by the front wheels on the horizontal curves. However this occurs only at excessively high speeds.

(c) The path traced by the wheels of a trailer in the case of trailer units, is also likely to be on either side of the central path of the towing vehicle, depending on the speed, rigidity of the universal joints and pavement roughness.

(d) In order to take curved path with larger radius and to have greater visibility at curve, the drivers have tendency not to follow the central path of the lane, but to use the outer side at the beginning of a curve.

(e) While two vehicles cross or overtake at horizontal curve there is a psychological tendency to maintain a greater clearance between the vehicles, than on straights for increase safety.

Thus the required extra widening of the pavement at the horizontal curves W_e depends on the length of wheel based of the vehicle l, radius of the curve negotiated R and the psychological factor which is a function of the speed of the vehicle and the radius of the curve.

It has been a practice therefore to provide extra width of pavement on horizontal curves when the radius is less than about 300 m.

Analysis of Extra Widening on Curves

The extra widening of pavement on horizontal curves is divided into two parts (i) mechanical and (ii) psychological widening.

Mechanical widening

The widening required to account for the off-tracking due to the rigidity of wheel based is called *mechanical widening* (W_m) and may be calculated as given below. Refer Fig. 4.25.

R_1 = radius of the path traversed by the outer rear wheel, m

R_2 = radius of the path traverse by the outer front wheel, m

W_m = off-tracking or the mechanical widening, m

l = length of wheel base, m

$$W_m = OC - OA = OB - OA = R_2 - R_1$$

From Δ OAB, $OA^2 = OB^2 - BA^2$

$$R_1^2 = R_2^2 - l^2$$

But $R_1 = R_2 - W_m$

$$(R_2 - W_m)^2 = R_2^2 - l^2$$

i.e., $R_2^2 - 2R_2W_m + W_m^2 = R_2^2 - l^2$

$$l^2 = W_m(2R_2 - W_m)$$

$$W_m = \frac{l^3}{2R_2 - W_m} \qquad (4.15)$$

$$= \frac{l^2}{2R} \ (approximately)$$

Here R is the mean radius of the curve. The mechanical widening calculated above is required for one vehicle negotiating a horizontal curve along one traffic lane. Hence in a road having 'n' traffic lanes, as 'n' vehicles can travel simultaneously, the total mechanical widening required is given by

$$W_m = \frac{nl^2}{2R} \qquad (4.16)$$

Psychological widening

Extra width of pavement is also provided for psychological reasons such as, to provide for greater maneuverability of steering at higher speeds, to allow for the extra space requirements for the overhangs of vehicles and to provide greater clearance for crossing and overtaking vehicles on the curves. Psychological widening is therefore important in pavements with more than one lane. An empirical formula has been recommended by IRC for finding the additional psychological widening 'W_{ps}' which is dependent on the design speed V of the vehicle and the radius R of the curve. The psychological widening is given by the formula :

$$W_{ps} = \frac{V}{9.5\sqrt{R}} \tag{4.17}$$

Hence the total widening We, m required on a horizontal curve is given by :

$$W_e = W_m + W_{ps}$$

i.e.,

$$W_e = \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}} \tag{4.18}$$

Hence n = number of traffic lanes.

l = length of wheel base of longest vehicle, m. The value of l may normally be taken as 6.1 m or 6.0 m for commercial vehicles, if not known.

V = design speed, kmph

R = radius of horizontal curve, m

The extra width recommended by the Indian Roads Congress for single and two lane pavements are given in Table 4.11.

Table 4.11 Extra width of pavement at horizontal curves

Radius of curve (m)	Upto 20	20 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

Note : For multi-lane roads, the pavement widening is calculated by adding half the extra width of two-lane roads to each lane of the multi-lane road.

Methods of introducing extra widening

The widening is introduced gradually, starting from the beginning of the transition curve or the tangent point (T.P.) and progressively increased at uniform rate, till the full value of designed widening 'W_e' is reached at the end of transition curve where full values of superelevation is also provided, as shown in Fig. 4.26. The full value of extra width W_e is continued throughout the length of the circular curve and then decreased gradually along the length of transition curve. Usually the widening is equally distributed i.e., $W_e/2$ each on inner and outer sides of the curve. But on sharp curves of hill roads the extra widening W_e may be provided in full on inside of the curve.

On horizontal circular curves without transition curves, two-thirds the widening is provided at the end of the straight section, i.e., before the start of the circular curve and the remaining one-third widening is provided on the circular curve beyond the tangent point as in the case of superelevation. In such cases, the widening is provided on the inside of the curve. Refer Fig. 4.27.

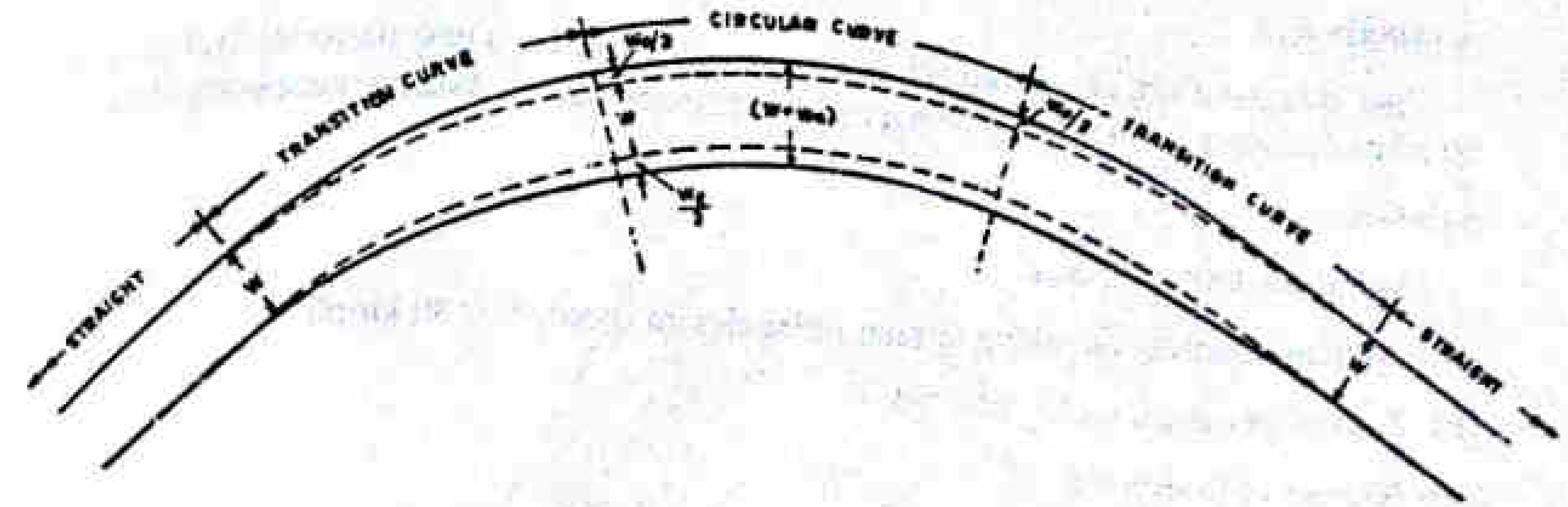


Fig. 4.26 Extra Widening of Pavement on Horizontal Curve

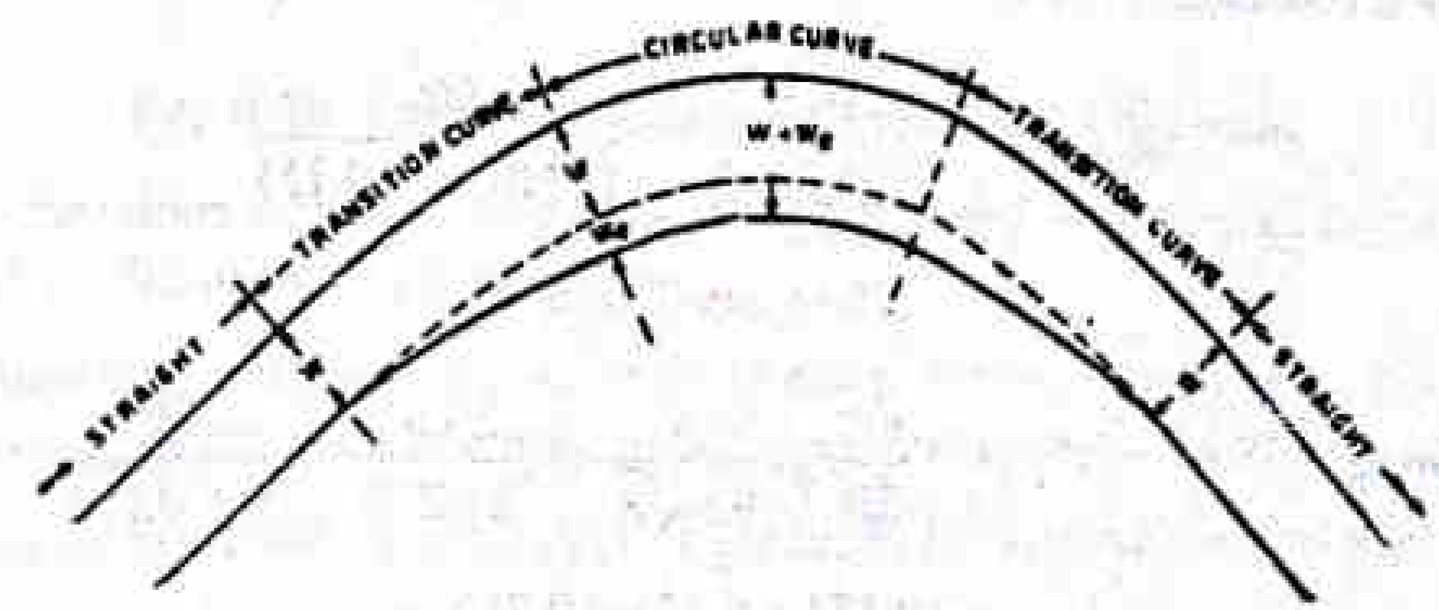


Fig. 4.27 Widening of Pavement on Sharp Curve

Example 4.14

Calculate the extra widening required for a pavement of within 7m on a horizontal curve of radius 250 m if the longest wheel base of vehicle expected on the road is 7.0 m. Design speed is 70 kmph. Compare the value obtained with IRC recommendations.

Solution

Extra widening required $W_e = W_m + W_{ps}$

$$= \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}} \tag{4.18}$$

Hence, n = 2 (two lanes for pavement width of 7.0 m)

l = 7.0

R = 250 m

V = 70 kmph

$$W_e = \frac{2\times 7^2}{2\times 250} + \frac{70}{9.5\sqrt{250}}$$

$$= 0.196 + 0.466 = 0.662 \text{ m}$$

The IRC recommends extra widening of 0.6 m when the radius of the curve is 101 to 300 m. (See Table 4.11)

Example 4.15

Find the total width of a pavement on a horizontal curve for a new national highway to be aligned along a rolling terrain with a ruling minimum radius. Assume necessary data.

Solution

Assume the following data :

(i) National highway on rolling terrain, ruling design speed, V = 80 kmph

(ii) Normal pavement width, W = 7.0

(iii) Number of lanes n = 2

(iv) Wheel base of the truck l = 6 m

(v) Maximum value of superelevation e = 0.07
and skid resistance f = 0.15

$$R_{ruling} = \frac{V^2}{127\,(e+f)} = \frac{80^2}{127\,(0.07+0.15)}$$

$$= 229 \text{ m, say } 230 \text{ m}$$

Extra widening $$W_e = \frac{n\,l^2}{2R} + \frac{V}{9.5\sqrt{R}} = \frac{2\times6^2}{2\times230} + \frac{80}{9.5\sqrt{250}}$$

$$= 0.157 + 0.555 = 0.712 \text{ m}$$

Total pavement width on curve $= W + W_e = 7.0 + 0.71 = 7.71$ m

4.4.7 Horizontal Transition Curve

A transition curve has a radius which decreases from infinity at the tangent point to a designed radius of the circular curve. When a transition curve is introduced between a straight and circular curve, the radius of the transition curve decreases becomes minimum at the beginning of the circular curve. The rate of change of radius of the transition curve will depend on the equation of the curve or its shape.

Object of Providing Transition Curves

Suppose a curve of radius R takes off from straight road, and a vehicle travels on this road; then due to the centrifugal force which suddenly acts on the vehicle just after the tangent point, a sudden lateral jerk is felt on the vehicle. This not only causes discomfort to the passengers, but also makes it difficult to steer the vehicle safely. Refer Fig. 4.28. If a transition curve BC of length L_s is introduced between the straight AB and the circular curve CD of radius R, the centrifugal force will also be introduced gradually as the radius of the transition curve decreases gradually from infinity. The rate at which this force is introduced can be controlled by adopting suitable shape of the transition curve and by designing its length, so that the vehicle can have a smooth entry from the straight to the circular curve at the design speed.

A transition curve which is introduced between the straight and a circular curve will help also in gradually introducing the designed superelevation and the extra widening necessary.

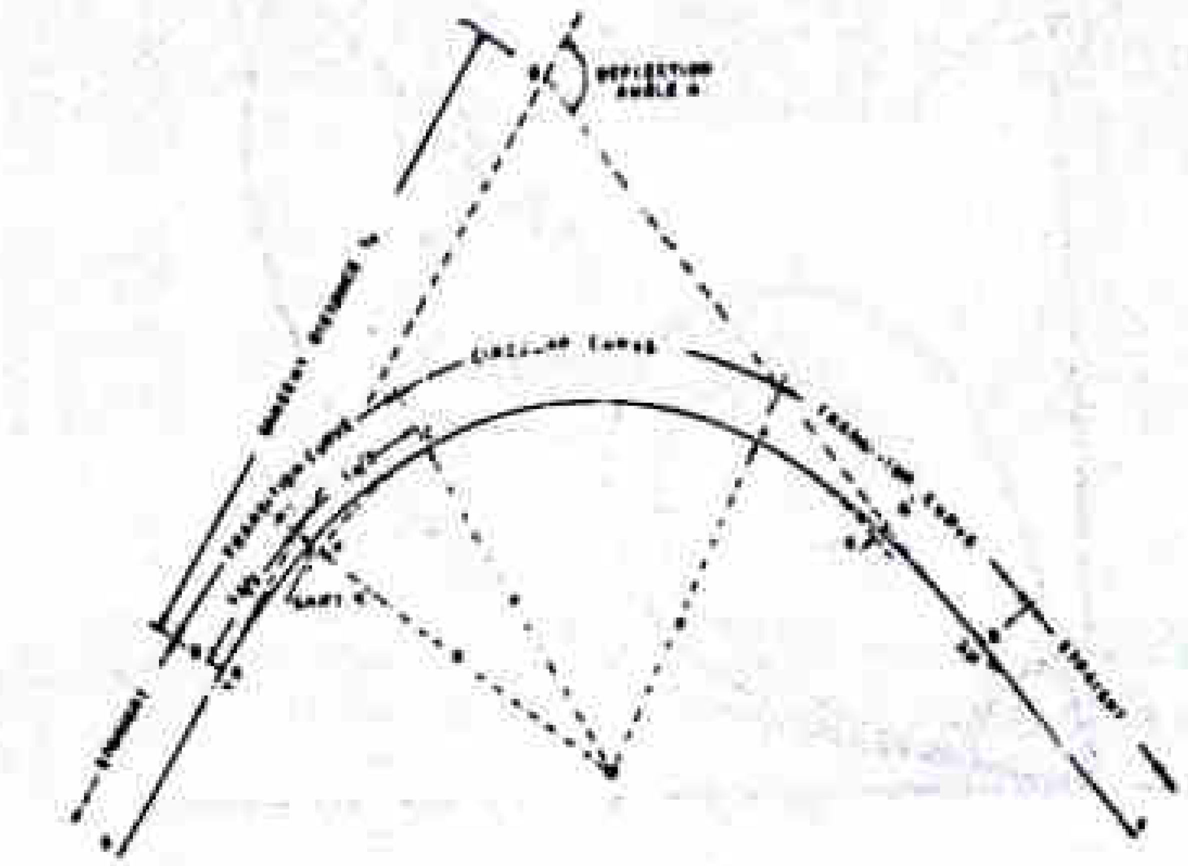


Fig. 4.28 Transition Curve in Horizontal Alignment

Thus the functions of transition curves in the horizontal alignment of highway may be summed up into the following points :

(a) to introduce gradually the centrifugal force between the tangent point and the beginning of the circular curve, avoiding a sudden jerk on the vehicle.

(b) to enable the driver turn the steering gradually for his own comfort and security.

(c) to enable gradual introduction of the designed superelevation and extra widening of pavement at the start of the circular curve.

(d) to improve the aesthetic appearance of the road.

In a good highway alignment it should be possible to maintain the design speed even on horizontal curves. The radius is first designed as discussed in article 4.3.4 and then a suitable shape of the transition curve is selected and its length is designed. *The ideal shape of a transition curve* should be such that the rate of introduction of centrifugal force or the rate of change of centrifugal acceleration should be consistent. This means that the radius of the transition curve should consistently decrease from infinity at the tangent point B (refer Fig. 4.28) to the radius R of the circular curve at point C, the end of the transition curve of length L_s. In an ideal transition curve the length L_s should be inversely proportional to the radius R i.e., ($L_s \propto 1/R$) or L_s R is a constant. The *spiral transition* fulfils this requirement.

Different Types of Transition Curves

The types of transition curves commonly adopted in horizontal alignment are :

(a) Spiral (also called *clothoid*)

(b) Lemniscate

(c) Cubic parabola

The general shapes of these three curves are shown in Fig. 4.29. All the three curves follow almost the same path upto deflection angle of 4°, and practically there is no significant difference even upto 9°. In all these curves, the radius decreases as the length

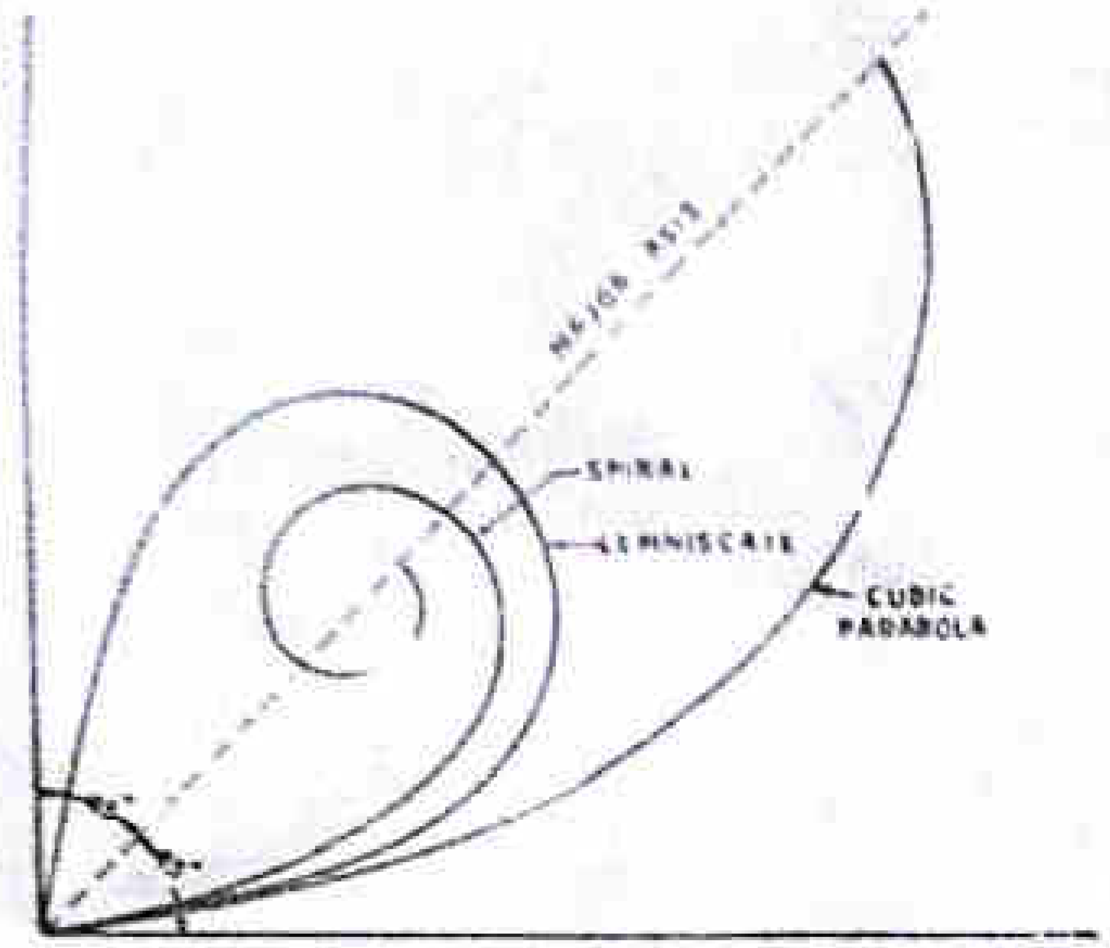


Fig. 4.29 Different Types of Transition Curves

increases. But the rate of change of radius and hence the rate of change of centrifugal acceleration is not constant in the case of lemniscate and cubic parabola, especially at deflection angles higher than 4°. In spiral curve the radius is inversely proportional to the length and the rate of change of centrifugal acceleration is uniform throughout the length of the curve. Thus the spiral fulfils the condition of an ideal transition curve.

The Indian Roads Congress recommends the use of the spiral as transition curve in the horizontal alignment of highways due to the following reasons :

(i) The spiral curve satisfies the requirements of an ideal transition.

(ii) The geometric property of spiral is such that the calculations and setting out the curve in the field is simple and easy.

The equation of the spiral may be written as :

$$L.R = L_S.R_C = \text{constant}$$

Therefore,
$$L = m\sqrt{\theta} \tag{4.19}$$

Here m is a constant equal to $\sqrt{2RL_s}$ and θ is the tangent deflection angle in radius.

Calculation of Length of Transition Curve

The length of transition curve is designed to fulfil three conditions, viz. : (i) rate of change of centrifugal acceleration to be developed gradually (ii) rate of introduction of the designed superelevation to be at a reasonable rate (iii) minimum length by IRC empirical formula.

(i) *Rate of change of centrifugal acceleration* : At the tangent point the centrifugal acceleration (v^2/R) is zero at the radius R is infinity. At the end of the transition the radius R has the minimum value R_m. Hence the centrifugal acceleration is distributed over a length L_s of the transition curve. The centrifugal acceleration should be developed at such a low rate that it will not cause discomfort for the passengers of a vehicle traveling at the design speed (v m/sec). It is evident that larger the length of transition, lower will be the rate at which the centrifugal acceleration is introduced.

Let the length of transition curve be L_s metre. If 't' is the time taken in seconds to traverse this transition length at uniform design speed of v m/sec, t = L_s/v. The maximum centrifugal acceleration of v^2/R is introduced in time 't' through the transition length L_s and hence the rate of change of centrifugal acceleration 'C' is given by

$$C = \frac{v^2}{R.t} = \frac{v^2}{R\frac{L_s}{v}}$$

$$C = \frac{v^3}{L_s.R}\ (\text{m/sec}^3) \tag{4.20}$$

The maximum allowable value of the rate of change of centrifugal acceleration without producing discomfort or undesirable oscillation, is dependent on the speed and varies inversely with the radius. The IRC has recommended the following equation for finding the value of C for the design speed V kmph :

$$C = \frac{80}{(75+V)}\ \text{m/sec}^3,\ [0.5 < C < 0.8] \tag{4.21}$$

i.e., the minimum and maximum values of C are limited to 0.5 and 0.8 respectively.

Once the value of 'C' is decided based on the design speed as given above the length of transition curve L_S can be calculated from the Eq. 4.20 which may be rewritten as :

$$L_s = \frac{v^3}{CR} \tag{4.22}$$

If the design speed is V kmph;

$$L_s = \frac{V^3}{(3.6)^3\,CR}$$

i.e.,
$$L_s = \frac{V^3}{46.5CR} = \frac{0.0215\,V^3}{CR} \tag{4.23}$$

Here,

L_s = length of transition curve, m

C = allowable rate of change of centrifugal acceleration, m/sec³ as given in Eq. 4.21.

R = radius of the circular curve, m.

(ii) *Rate of Introduction of superelevation* : In open country if a high value of superelevation is to be introduced, it is not desirable to raise the outer edge of a pavement at a larger rate than 1 in 150 relative to the grade of the centre line. Hence the length of transition curve should be atleast 150 times the total amount by which the outer edge of the pavement is to be raised with respect to the centre line. However, the transition curve length may be reduced by allowing an increased differential gradient of 1 in 100 in built up areas and 1 in 60 on hill roads. If the pavement is rotated about the inner edge and not the centre line, then the total lifting of outer edge with respect to inner edge has to be considered in calculating the length of transition curve required.

Let 'e' be the rate of superelevation designed as per Eq. 4.9 for a highway curve having normal pavement width W. Let 'W_e' be the extra widening provided at the circular curve so that the total width B of pavement = $(W + W_e)$ and the total raising of pavement with respect to the inner edge = e.B = e. $(W + W_e)$ = E. If it is assumed that the pavement is rotated about centre line after neutralizing the camber, (maintaining the vertical alignment of the centre line) then the maximum amount by which the outer edge is to be raised at the circular curve with respect to the centre = E/2. Hence allowing a rate of change of superelevation of 1 in N (where minimum value of N = 150 to 60 as discussed above), the length of transition curve L_s is given by :

$$L_s = \frac{EN}{2} = \frac{eN}{2}(W + W_e) \tag{4.24a}$$

However if the pavement is rotated about the inner edge, the length of transition curve is given by :

$$L_s = EN = eN(W + W_e) \tag{4.24b}$$

(iii) *By Empirical Formula* : According to the IRC standards, the length of horizontal transition curve L_s should not be less than the value given by the following equations for the terrain classifications :

(a) For plain and rolling terrain :

$$L_s = \frac{2.7V^2}{R} \tag{4.25a}$$

(b) For mountainous and steep terrains;

$$L_s = \frac{V^2}{R} \tag{4.25b}$$

The length of transition curve for the design should be the highest of the three values mentioned above. Therefore, the design steps are given below :

(a) Find the length of transition curve based on allowable rate of change of the centrifugal acceleration (Eq. 4.21 and 4.22 or 4.23).

(b) Find the length of transition curve based on rate of change of superelevation (Eq. 4.24 or 4.25).

(c) Check for the minimum required value of L_s as per Eq. 4.25a or 4.25b.

(d) Adopt the highest value of L_s given by (a), (b) and (c) above as the design length of transition curve.

The minimum length of transition curves for various values of radius of curve and design speeds recommended by the IRC for plain and rolling terrains and also for mountainous and steep terrains are given in Table 4.12.

Table 4.12 Minimum transition length for different speeds and curve radii

Plain and rolling terrain							Mountainous and steep terrain					
Curve Radius R(m)	Design speed (kmph)						Curve Radius R(m)	Design speed (kmph)				
	100	80	65	50	40	35		50	40	30	25	20
	Transition length, m							Transition length, m				
45	-	-	-	-	NA	70	14	-	-	-	NA	30
60	-	-	-	NA	75	55	20	-	-	-	35	20
90	-	-	-	75	50	40	25	-	-	NA	25	20
100	-	-	NA	70	45	35	30	-	-	30	25	15
150	-	-	80	45	30	25	40	-	NA	25	20	15
170	-	-	70	40	25	20	50	-	40	20	15	15
200	-	NA	60	35	25	20	55	-	40	20	15	15
240	-	90	50	30	20	NR	70	NA	30	15	15	15
300	NA	75	40	25	NR	-	80	55	25	15	15	NR
360	130	60	35	20	-	-	90	45	25	15	15	-
400	115	55	30	20	-	-	100	45	20	15	15	-
500	95	45	25	NR	-	-	125	35	15	15	NR	-
600	80	35	20	-	-	-	150	30	15	15	-	-
700	70	35	20	-	-	-	170	25	15	NR	-	-
800	60	30	NR	-	-	-	200	20	15	-	-	-
900	55	30	-	-	-	-	250	15	15	-	-	-
1000	50	30	-	-	-	-	300	15	NR	-	-	-
1200	40	NR	-	-	-	-	400	15	-	-	-	-
1500	35	-	-	-	-	-	500	NR	-	-	-	-
1800	30	-	-	-	-	-						
2000	NR	-	-	-	-	-						

Note : NA – Not applicable; NR – Transition not required

The length of transition curve L_s required on a horizontal highway curve therefore depends upon the following factors :

(i) Radius of circular curve, R

(ii) Design speed, V

(iii) Allowable rate of change of centrifugal acceleration, C (which is also dependent on the design speed)

(iv) Maximum amount of superelevation, E which depends on the maximum rate of superelevation, *e* and the total width of the pavement, B at the horizontal curve

(v) Whether the pavement cross section is rotated about the inner edge or the centre line, after the elimination of the camber.

(vi) Allowable rate of introduction of superelevation, which depends on the terrain, location and environmental conditions of the site.

Setting out of transition curve

When transition curves are to be provided on both ends of a circular curve, the following procedure may be adopted. Refer Fig. 4.28. Let PCDQ be the original circular curve of radius R. PP' and QQ' are equal to the shift S of the transition curve given by the formula :

$$S = \frac{L_s^2}{24R} \tag{4.26}$$

where L_s is the length of transition curve and R is the radius of the circular curve. B P C and E Q' D are the two transition curves, each of length L_s and C D is the shifted circular curve. The length of BP' and P'C are approximately equal to $L_s/2$. The points B and E remain as tangent points to the new compound curve B P' C D Q' E.

In order to set out the transition spiral, the design details such as the radius of the circular curve R, length of transition curve L_s, total deviation angle Δ, tangent deviation angle of the transition θ_s, central angle of circular arc Δ_c, tangent distance, apex distance etc. are determined. The curve may be laid either by off-set method or by polar deflection angle method. The details of calculating the off sets/deflection angles and setting out the curve in the field are not given here; they are available in the books so Surveying.

Example 4.16

Calculate the length of transition curve and the shift using the following data :

Design speed = 65 kmph

Radius of circular curve = 220 m

Allowable rate of introduction of superelevation (pavement rotated about the centre line) = 1 in 150

Pavement width including extra widening = 7.5 m

Solution

(a) Length of transition curve L_s as per allowable rate of centrifugal acceleration C :

Allowable rate of change of centrifugal acceleration as per Eq. 4.21,

$$C = \frac{80}{(75+V)} = \frac{80}{(75+65)} = 0.57, \text{m/sec}^3$$

This value is between 0.5 and 0.8 and hence accepted.

$$L_s = \frac{0.0215\, V^3}{C\,R} = \frac{0.0215 \times 65^3}{0.57 \times 220} = 47.1 \text{ m}$$

(b) Length L_s by allowable rate of introduction of superelevation E :

Superelevation rate $$e = \frac{V^2}{225\,R} = \frac{65^2}{225 \times 220} = 0.085$$

As this value is greater than the maximum allowable rate of 0.07, limit the value of e = 0.07. Check the safety against transverse skidding for the design speed of 65 kmph :

$$f = \frac{V^2}{127\,R} - e = \frac{65^2}{127 \times 220} - 0.07$$

$$= 0.15 - 0.07 = 0.08$$

As this value of f is less than the allowable value of 0.15, the superelevation rate of 0.07 is safe for the design speed of 65 kmph.

Total width of the pavement at the curve, B = 7.5 m

Total raise of outer edge of pavement with respect to the centre line

$$= \frac{E}{2} = \frac{e\,B}{2} = \frac{0.07 \times 7.5}{2} = 0.26 \text{ m}$$

Rate of introduction of superelevation, 1 in N = 1 in 150

$$L_s = \frac{E\,N}{2} = 0.26 \times 150 = 39 \text{ m}$$

(c) Minimum value of L_s as per IRC (Eq. 4.25)

$$= \frac{2.7\,V^2}{R} = \frac{2.7 \times 65^2}{220} = 51.9 \text{ m}$$

Adopt the highest value of the three i.e., 51.9 or say 52 m as the design length of transition curve.

$$\text{Shift } S = \frac{L_s^2}{24R} = \frac{52^2}{24 \times 220} = 0.51 \text{ m}$$

Example 4.17

A national Highway passing through rolling terrain in heavy rain fall area has a horizontal curve of radius 500 m. Design the length of transition curve assuming suitable data.

Solution

For a National Highway on rolling terrain, the following data may be assumed as per standard practice :

Design speed, V = 80 kmph

Normal pavement width, W = 7.0 m

Allowable rate of change of centrifugal acceleration, (range of value 0.5 to 0.8)

$$C = \frac{80}{(75+V)} = \frac{80}{75+80} = 0.52$$

As the value of C is between 0.5 and 0.8 it is accepted for design.

Allowable rate of introduction of superelevation = 1 in 150, pavement to be rotated about the inner edge to effect better drainage in heavy rain fall area.

(a) Length of transition curve by rate of change of centrifugal acceleration :

$$L_s = \frac{v^3}{C\,R} = \frac{0.0215\,V^3}{C\,R} = \frac{0.0215 \times 80^3}{0.52 \times 500} = 42.3 \text{ m}$$

(b) Length of transition curve by the rate of introduction of superelevation :

$$e = \frac{V^2}{225\,R} = \frac{80^2}{225 \times 500} = 0.057 \ (< 0.07, \text{ O.K.})$$

Extra widening at curve (assuming two lanes and wheel base of 6 m)

$$W_e = \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}} = \frac{2 \times 6^2}{2 \times 500} + \frac{80}{9.5\sqrt{500}} = 0.45 \text{ m}$$

Total width of pavement $= B = 7.0 + 0.45 = 7.45$ m

$$L_s = 7.45 \times 0.057 \times 150 = 63.7 \text{ m}$$

(c) Check for minimum value of L_s by Eq. 4.25 a,

$$L_s = \frac{2.7\,V^2}{R} = \frac{2.7 \times 80^2}{500} = 34.6 \text{ m}$$

Adopt the highest of the above three values = 63.7 say, 64 m. Therefore, the design length of transition curve is 64 m.

4.4.8 Set-back Distance on Horizontal Curves

In the design of horizontal alignment, the sight distance along the inner side of the curves should be considered. Where there are sight obstruction like buildings, cut slopes, or tree on the inner side of the curves, either the obstruction should be removed or the alignment should be changed in order to provide adequate sight distance. It may some times be possible to make some adjustments in the normal highway cross section to make up small deficiencies in sight distance. If it is not possible to provide the adequate sight distance on curves on existing roads, regulatory and cautionary signs should be installed to control the traffic suitably. In case of new highways for the design speed and distance requirements, the actual condition in the alignment should be checked and necessary adjustments be made in a manner most fitting to provide adequate sight distance. Specific study is usually necessary for each site condition.

As discussed in Art. 4.3, the absolute minimum sight distance which is the safe stopping sight distance should be available at every section of the highway, from safety point. Thus it is essential that in horizontal alignment, special care should be taken to provide for the stopping sight distance; these values may be adopted as given in Table 4.5 for the design speed. Overtaking sight distance requirements are given in Table 4.7. The *clearance distance* or *set back distance* required from the centre line of a horizontal curve to an obstruction on the inner side of the curve to provide adequate sight distance depends upon the following factors :

(i) required sight distance, S

(ii) radius of horizontal curve, R

(iii) length of the curve, L_c which may be greater or lesser than S.

Refer Fig. 4.31. Let C be the obstruction to vision on the inner side of a horizontal highway curve of radius R, ABC the line of sight and arc AB be the sight distance S.

(a) $L_c > S$

Let the length of curve L_c be greater than the sight distance S. The angle subtended by the arc length S at the centre be α. On narrow roads such as single lane roads, the sight distance is measured long the centre line of the road and the angle subtended at the centre, α is equal to S/R radians.

Therefore half central angle is given by :

$$\frac{\alpha}{2} = \frac{S}{2R} \text{ radians} = \frac{180\,S}{2\pi R} \text{ degrees}$$

The distance from the obstruction to the centre is R cos α/2. Therefore the set-back distance, m required from the centre line is given by :

$$m = R - R\cos\frac{\alpha}{2} \qquad (4.27a)$$

In the case of wide roads with two or more lanes, if d is the distance between the centre line of the road and the centre line of the inside lane in metre, the sight distance is measured along the middle of the inner side lane and the set-back distance, m' is given by :

$$m' = R - (R - d)\cos\frac{\alpha'}{2} \qquad (4.27b)$$

where $\frac{\alpha'}{2} = \frac{180\,S}{2\pi(R-d)}$ degrees

(b) $L_c < S$

If the sight distance required is greater than the length of curve L_c, then the angle α subtended at the centre is determined with reference to the length of circular curve, L_c and the set-back distance is worked out in two parts as given below : See Fig. 4.33.

$$\frac{\alpha}{2} = \frac{180\,L_c}{2\pi(R-d)} \text{ degrees}$$

$$m' = R - (R - d)\cos\frac{\alpha'}{2} + \frac{(S - L_c)}{2}\sin\frac{\alpha'}{2} \qquad (4.28)$$

The clearance of obstruction upto the set-back distance is important when there is cut slope on the inner side of the horizontal curve. The method of calculating the set-back distance is illustrated in Examples 4.19 and 4.20.

4.4.9 Curve Resistance

The automobiles are steered by turning the front wheels, but the rear wheels do not turn. When a vehicle driven by rear wheels moves on a horizontal curve, the direction of rotation of rear and front wheels are different, as shown in Fig. 4.30 and so there is some loss in the tractive force.

A and B are the rear driving wheels which give a tractive force T in the direction PQ. The front wheels C and D are turned so as to steer the vehicle along a horizontal curve, the tangential direction of which is RS. Hence the tractive force available in this direction = T cos α which will be less than the actual tractive force, T applied. Obviously when the turning radius is sharp, the turning angle will be high and the value of T cos α would decrease. Thus the loss of tractive force due to turning of a vehicle on a horizontal curve, which is termed as *curve resistance* will be equal to (T − T cos α) or T (1 − cos α) and will depend on the turning angle α.

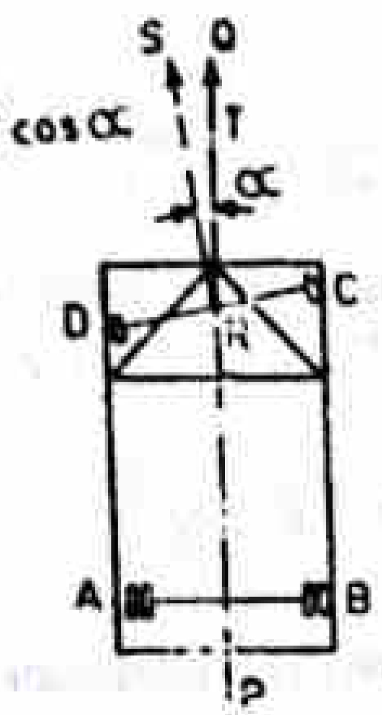


Fig. 4.30 Curve Resistance for Turning Vehicle

Turning along the horizontal curve is effected due to the lateral friction developed between the front wheels and the pavement. At sharp curves, if the speed is high there may be even sliding along the tangential direction PQ. Thus while a vehicle driven by rear wheels turns along a horizontal curve, there is increased resistance and if the same speed as on straight is to be maintained, a higher tractive effort is needed. But in the vehicles with front driving wheels, this problem does not exist. Most of the heavy commercial vehicles have rear driving wheels and hence on sharp curves the additional curve resistance should also be considered while designing the geometric features of highways. This problem of curve resistance is acute on hill roads as the curves are often sharp and in addition, the roads have steep gradients. The compensation in gradient needed for such a case has been explained later in Art. 4.5.2.

4.4.10 General Examples on Horizontal Alignment

Example 4.18

While aligning a highway in a built up area, it was necessary to provide a horizontal circular curve of radius 325 metre. Design the following geometric features :

(i) Superelevation

(ii) Extra widening of pavement

(iii) Length of transition curve

Data available are

Design speed = 65 kmph, Length of wheel base of largest truck = 6 m, Pavement width = 10.5 m

Solution

(i) Superelevation rate, e

From practical considerations of mixed traffic conditions, superelevation to fully counteract centrifugal force should be designed with 75% of design speed. Here radius R = 325 m.

$$e = \frac{V^2}{225\,R} = \frac{65^2}{225 \times 325} = 0.058$$

As this value is less than 0.07, it is safe for the design speed.

Hence provide superelevation rate = 0.058.

(ii) Extra widening of pavement, W_e

$$W_e = \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}}$$

n = 3 as the pavement width is 10.5 m

Wheel base = 6 m

$$W_e = \frac{3 \times 6^2}{2 \times 325} + \frac{65}{9.5\sqrt{325}}$$

$$= 0.166 + 0.380 = 0.546, \text{ say, } 0.55 \text{ m}$$

(iii) Length of transition curve, L_s

(a) By rate of change of centrifugal acceleration :

Allowable rate of change of centrifugal acceleration C is given by :

$$C = \frac{80}{75 + V} = \frac{80}{76 + 65} = 0.57 \text{ m/sec}^3$$

(As this value is between 0.5 and 0.8, accepted for design)

$$L_s = \frac{0.0215\,V^3}{C\,R} = \frac{0.0215 \times 65^3}{0.57 \times 325} = 31.9 \text{ m}$$

(b) By rate of introduction of superelevation, E : Total superelevation, E = B × e.

Total pavement width including extra widening on curve,

$$B = W + W_e = 10.5 + 0.55 = 11.05 \text{ m}$$

Superelevation rate, e = 0.058

$$E = 11.05 \times 0.058 = 0.64 \text{ m}$$

Assuming that superelevation is provided by rotating about the centre line, the total superelevation to be distributed along the length of transition curve = E/2. The rate of introduction of superelevation may be taken as 1 in 100, being built up area.

Length of transition curve $L_s = \frac{0.64}{2} \times 100 = 32$ m

(c) By IRC formula, the minimum length

$$L_s = \frac{2.7\,V^2}{R} = \frac{2.7 \times 65^2}{325} = 35.1 \text{ m}$$

Adopting highest of the above three values, length of transition curve L_s = 35 m.

Example 4.19

A State Highway passing through a rolling terrain has a horizontal curve of radius equal to the ruling minimum radius.

(i) Design all the geometric features of this curve, assuming suitable data.

(ii) Specify the minimum set-back distance from the centre line of the two lane highway on the inner side of the curve up to which the buildings etc. obstructing vision should not be constructed so that Intermediate sight distance is available throughout the circular curve. Assume the length of circular curve greater than the sight distance.

Solution

The various geometric elements to be designed are

(a) Ruling minimum radius

(b) Superelevation

(c) Extra widening

(d) Length of transition curve

(e) SSD, ISD and set-back distance

(a) Ruling minimum radius of curve for ruling design speed of 80 kmph :

$$R_{ruling} = \frac{V^2}{127\,(e+f)} = \frac{80^2}{127\,(0.07+0.15)}$$

$$= 229 \text{ m, say } 230 \text{ m}$$

(b) Design value of superelevation :

$$e = \frac{V^2}{225\,R} = \frac{80^2}{225\times 230} = 0.124$$

As the value is higher than the maximum superelevation of 0.07, limit the value of e to 0.07. The curve should be safe for the full speed of 80 kmph as the ruling minimum radius has been adopted. However check the transverse skid resistance developed :

$$f = \frac{V^2}{127\,R} - e = \frac{80^2}{127\times 230} - 0.07 = 0.149$$

(Less than 0.15 and hence safe)

(c) Assume two lane pavement, i.e., n = 2 and l = 6 m. Extra widening of pavement,

$$W_e = \frac{nl^2}{2R} + \frac{V}{0.5\sqrt{R}}$$

$$= \frac{2\times 6^2}{2\times 230} + \frac{80}{9.5\sqrt{230}} = 0.157 + 0.555 = 0.712 \text{ m}$$

Provide an extra width of 0.71 m and a total width of pavement B = 7.71 m.

(d) Length of transition curve is designed by calculating the values based on (i) rate of change of centrifugal acceleration C (ii) rate of introduction of the amount of superelevation E and (iii) minimum length formula; the highest of three values is adopted at the design length L_S.

(i)
$$C = \frac{80}{75+V} = \frac{80}{75+80} = 0.52$$

(as this is within the range 0.5 to 0.8, the value is acceptable for design).

$$L_s = \frac{0.0215\,V^3}{C\,R} = \frac{0.0215\times 80^3}{0.52\times 230} = 92 \text{ m}$$

(ii) Total amount of superelevation E i.e., the raising of the outer edge of the pavement with respect to inner edge = B × e = 7.71 × 0.07 = 0.54 m. As the terrain is rolling, assume the pavement to be rotated about the centre at a rate of 1 in 150.

$$L_s = \frac{E}{2} \times N = \frac{0.54\times 150}{2} = 40.5 \text{ m}$$

(iii) Minimum value of L_s as per IRC is given by :

$$L_s = \frac{2.7\,V^2}{R} = \frac{2.7\times 80^2}{230} = 75.1 \text{ m}$$

Adopting the highest of the three values, design length of transition curve = 92 m.

(e) Intermediate Sight Distance = 2 SSD

$$= 2\left[0.278\ Vt + \frac{V^2}{254\,f}\right]$$

$$= 2\left[0.278\times 80\times 2.5 + \frac{80^2}{254\times 0.35}\right] = 2\times 127.6 = 255 \text{ m}$$

(f) Refer Fig. 4.31. The length of circular curve is assumed greater than the desired sight distance SD. The minimum clearance or set-back distance needed m = CD and half the central angle $\alpha'/2$ = angle AOD.

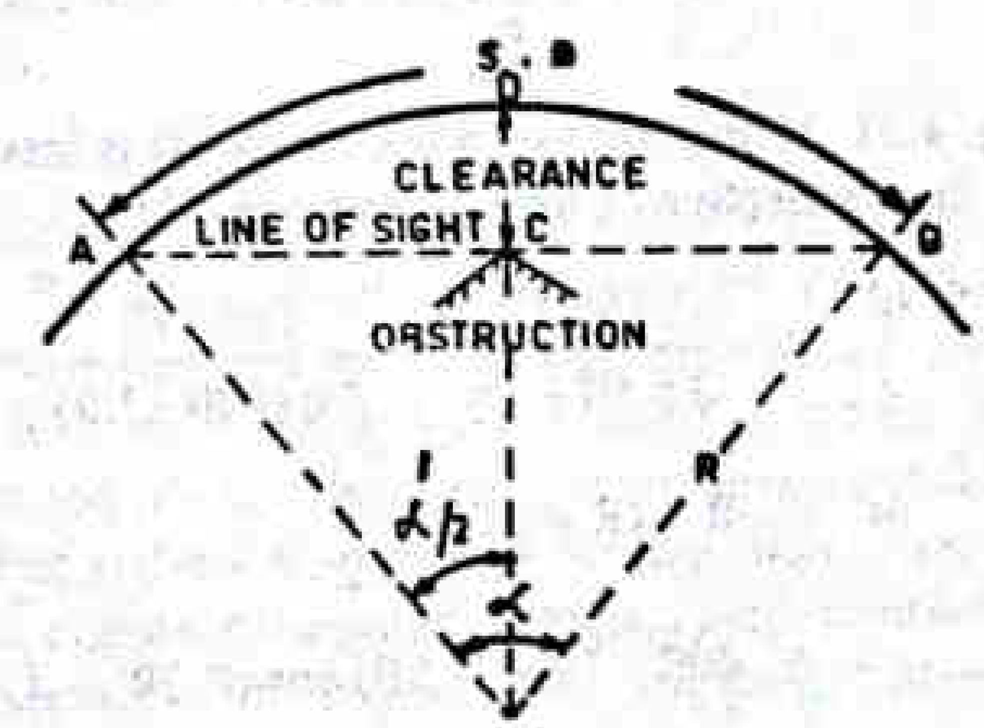


Fig. 4.31 Clearance on curve or Set-back Distance for Desired Sight Distance (Example 4.19)

The distance d between the centre line of the pavement and the centre line of the inside lane may be taken as one-fourth the width of pavement at the curve (being a two-lane pavement) = 7.71/4 = 1.93 m.

$$\frac{\alpha'}{2} = \frac{180\,S}{2\pi\,(R-d)} = \frac{180 \times 255}{2\pi\,(230-1.93)} = 32°$$

$$\text{Set-back distance } m' = R - (R-d)\cos\frac{\alpha'}{2}$$

$$= 230 - (230 - 1.93)\cos 32° = 36.6 \text{ m}$$

Therefore the minimum set-back distance or clearance required to provide a clear vision for an ISD of 255 m is 36.6 m.

Example 4.20

There is a horizontal highway curve of radius 400 m and length 200 m on this highway. Computer the set-back distances required from the centre line on the inner side of the curve so as to provide for

(a) stopping sight distance of 90 m

(b) safe overtaking sight distance of 300 m.

The distance between the centre lines of the road and the inner lane is 1.9 m.

Solution

(a) Refer Fig. 4.32. The stopping sight distance (SSD) of 90 m is less than the circular curve length of 200 m

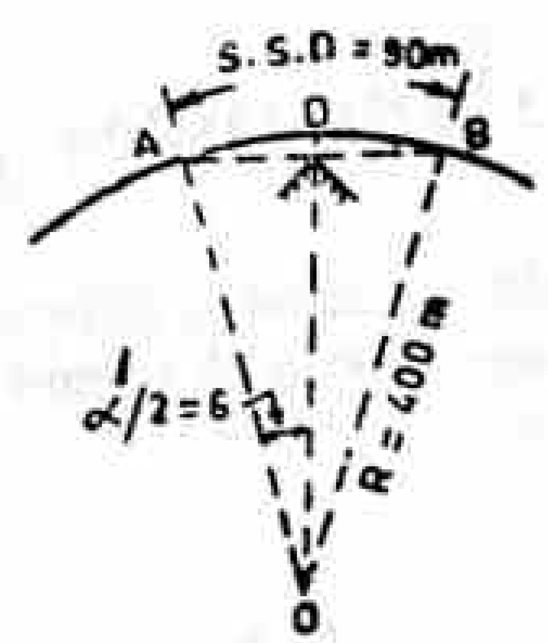


Fig. 4.32 Minimum Set-back when SD is less than Length of Curve (Example 4.20a)

$$\frac{\alpha'}{2} = \frac{180\,S}{2\pi\,(R-D)} = \frac{180 \times 90}{2\pi\,(400-1.9)} = 6°\,29'$$

$$m' = R - (R-d)\cos\frac{\alpha'}{2}$$

$$= 400 - (400 - 1.9)\cos 6°\,29' = 4.4 \text{ m}$$

Required clearance from the centre line to provide SSD of 90 m is 4.4 m.

(b) Refer Fig. 4.33. The overtaking sight distance of 300 m is greater than circular curve length which is 200 m. Therefore the required set-back distance is CF = (CG + GF) and is given by Eq. 4.28.

$$S = 300 \text{ m},\ L_c = 200 \text{ m},\ R = 400 \text{ m},\ d = 1.9 \text{ m}$$

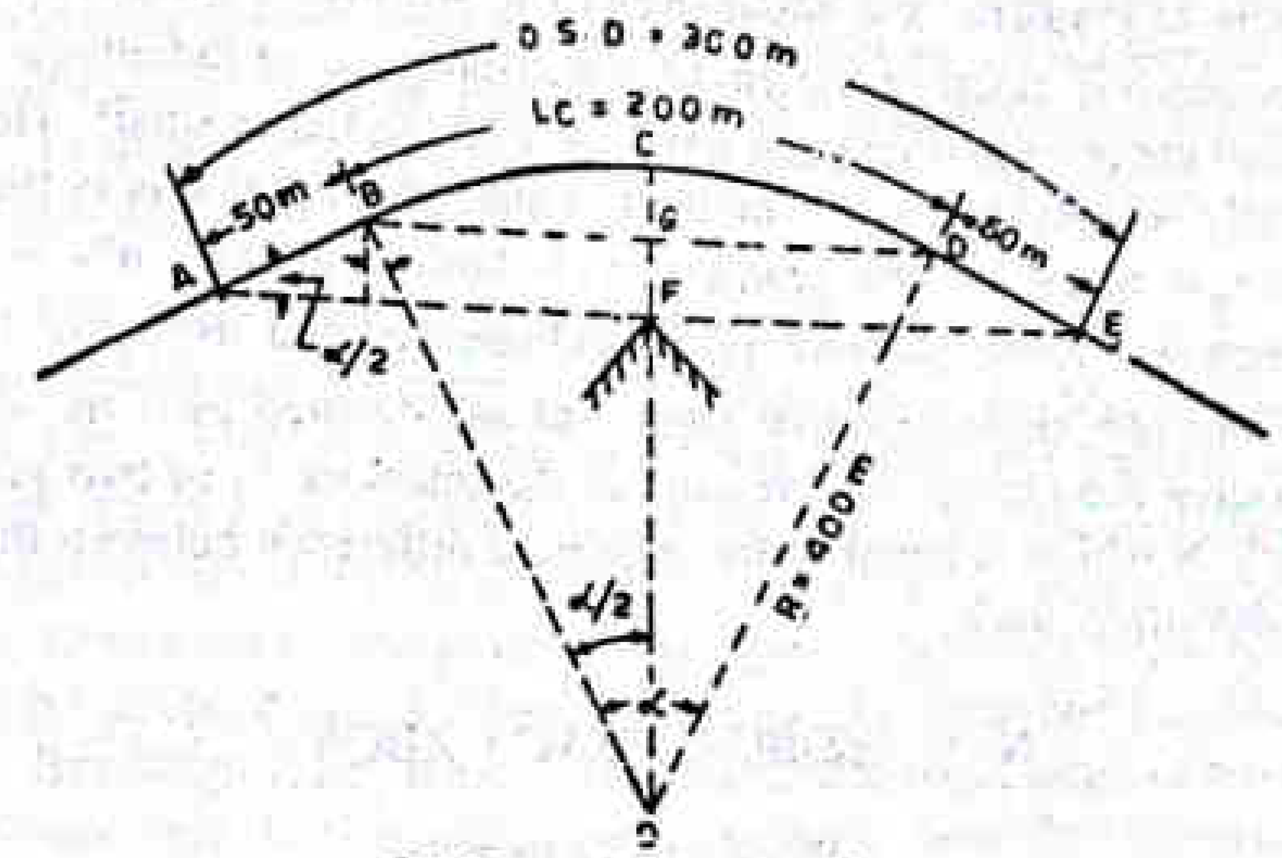


Fig. 4.33 Minimum Set-back Distance when SD is Greater than Length of Curve (Example 4.20b)

$$\frac{\alpha'}{2} = \frac{180\,L_c}{2\pi\,(R-d)} = \frac{180 \times 200}{2\pi\,(400-1.9)} = 14.39°$$

$$\text{Set-back distance } m' = CF = CG + GF$$

$$= R - (R-d)\cos\frac{\alpha'}{2} + \frac{(S-L_c)}{2}\sin\frac{\alpha'}{2}$$

$$= 400 - (400 - 1.9)\cos 14.39° + \frac{(300-200)}{2}\sin 14.39°$$

$$= 14.4 + 12.4 = 26.8$$

Minimum set-back distance required from the centre line of the roads on the inner side of the pavement to provide an OSD of 300 m = 27 m.

4.5 DESIGN OF VERTICAL ALIGNMNET

4.5.1 General

While aligning a highway it is the general practice to follow the general topography or profile of the land. But the natural ground may be level only at some places and otherwise the ground may have slopes of varying magnitudes. Hence the vertical profile of a road would have level stretches as well as slopes or grades. In order to have smooth vehicle movements on the roads, the changes in the grade should be smoothened out by the vertical curves. The vertical alignment is the elevation or profile of the centre line of the road. The vertical alignment consists of grades and vertical curves, and it influences the vehicle speed, acceleration, deceleration, stopping distance, sight distance and comfort in vehicle movements at high speeds.

4.5.2 Gradient

Gradient is the rate of rise or fall along the length of the road with respect to the horizontal. It is expressed as a ratio of 1 in *x* (1 vertical unit to *x* horizontal units). Some times the gradient is also expressed as a percentage, n i e, n in 100.

When the angle of gradient, α is small (Refer Fig. 4.34a) the gradient which is 1 in α or tan α is approximately equal to the circular measure or α in radians (α^c). All angles within the practical range of gradients on roads may be treated as small. Hence gradients which are generally represented as 'n' percent, would mean that this is the value of the tangent of the angle made by the gradient with horizontal, i.e., n% = tan α. The ascending gradients are given positive signs and are denoted as $+ n_1$, $+ n_2$ etc., and descending gradients are given negative signs and are denoted as $- n_3$, $- n_4$ etc. The angle which measures the change of direction at the intersection of two grades is called the *deviation angle* N which is equal to the algebraic difference between the two grades. In Fig. 4.34b the deviation angle,

$$N = \angle DBC = \angle BAC + \angle BCA$$

$$= + n_1 - (- n_2) = n_1 + n_2$$

where n_1 is ascending gradient of AB and $- n_2$ the descending gradient of BC.

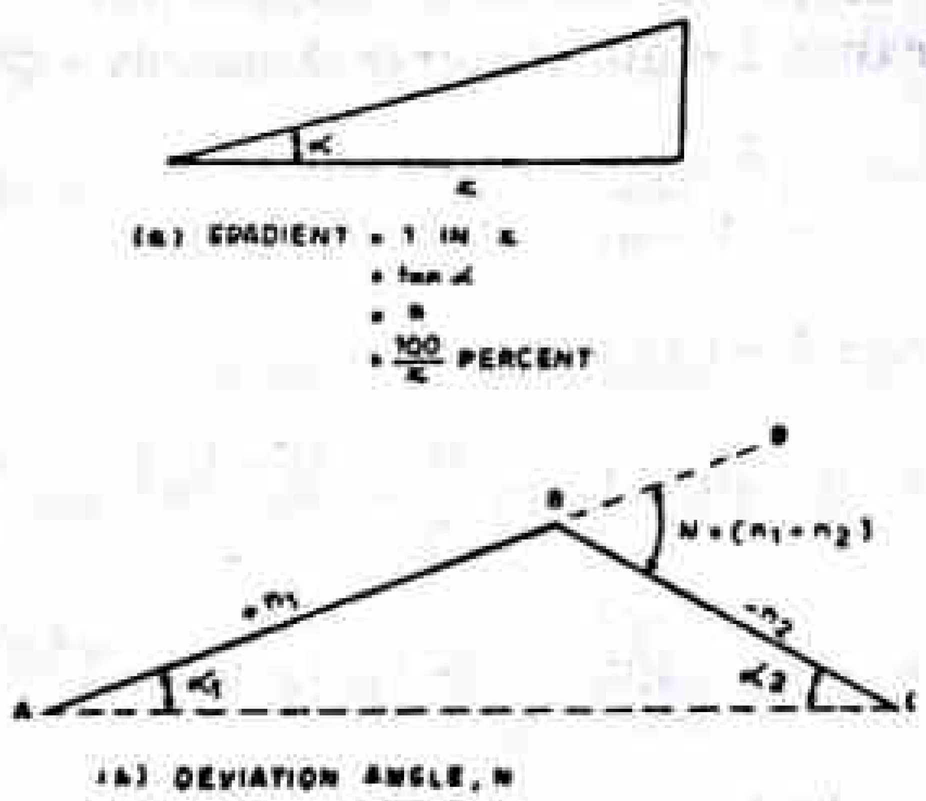


Fig. 4.34 Measure of Gradients

While aligning a highway, the gradient is decided for designing the vertical curve. Very steep gradients are avoided as it is not difficult to climb the grade, but also the vehicle operation cost is increased. The engineer has to consider all aspects such as construction costs, practical problems in construction at the site and the vehicle operation cost in such alternative proposals before finalising the gradients.

Gradients are divided into the following categories :

Ruling gradient

Limiting gradient

Exceptional gradient

Minimum gradient

The terms are explained below :

Ruling gradient is the maximum gradient within which the designer attempts to design the vertical profile of a road. Gradients up to the ruling gradient are adopted as a normal course in design of vertical alignment and accordingly the quantities of cut and fill are balanced. Hence ruling gradient is also known as design gradient. However flatter gradients may be preferred where ever practicable.

The selection of ruling gradient for the purpose of design is a complex job as several factors such as type of terrain, the length of the grade, the speed, pulling power of vehicles and presence of horizontal curves are considered. In flat terrain or plain country it may be possible to adopt a flat gradient : but in hill roads it may not be economical or some time not even possible to adopt the same gradient because of large difference in levels to be covered in short length of road.

A vehicle which travels with a certain speed on a level ground, with the same tractive effort put in, would lose speed at grades; the speed would steadily decrease with increase in length of grade. With the maximum pulling power, the vehicle would be able to sustain the same speed even on long sections only up to a certain gradient. This is when the maximum power developed by the engine is equal to the power required to overcome the resistances to motion on the grade at this speed. Therefore this gradient, is the one which should be adopted at a ruling gradient by the designer for this vehicle and the design speed. But the problem is not so simple as different vehicles have different values of hauling power and varying tractive resistances and the commercial vehicles in particular have to carry different amount of load. Further in India due consideration is to be given to the pulling power of animal drawn vehicle, especially the bullock carts.

Thus it is not possible to lay down precise standards of ruling gradient applicable for the mixed traffic and for the country as a whole.

The IRC has recommended ruling gradient values of 1 in 30 on plain and rolling terrain, 1 in 20 on mountainous terrain and 1 in 16.7 on steep terrain.

Where topography of a place compels adopting steeper gradients than ruling gradients, *limiting gradients* are used in view of enormous increase in cost in constructing roads with gentler gradients. However the length of continuous grade line steeper than ruling value should be limited. On rolling terrain and on hill roads, it may be frequently necessary to exceed ruling gradient and adopt limiting gradient; but care should be taken to separate such stretches of steep gradients by providing either a level road or a road with easier grade.

In some extra ordinary situations it may be unavoidable to provide still steeper gradients atleast for short stretches and in such cases the steeper gradient upto *exceptional gradients* may be provided. However the exceptional gradient should be strictly limited only for short stretches not exceeding about 100 m at a stretch.

The maximum length of ascending gradient which a loaded truck can operate without undue reduction in speed is called *critical length of grade* for a design. A reduction in speed of about 25 kmph may be considered reasonable limit. The critical length for design depends on several factors such as size, power, load and grade ability data of the truck, its initial speed at the beginning of the ascending grade and the desirable limit of the minimum speed at the end of the grade so as to avoid unreasonable interference with the movement of other vehicles. The critical length of ascending gradients should therefore be limited to lower values at steeper gradients.

The maximum values of gradients recommended by the IRC at different terrains are given in Table 4.13.

The road can be level, with little or no gradient. In such cases there will be problems of drainage. Though the surface water can be drained off to the side drains by providing proper camber on the pavement surface and cross slope on shoulders, a certain longitudinal slope is essential, to drain the water along the side drains, depending on the surface of the drains.

Table 4.13 Gradients for Roads in Different Terrains

Terrain	Ruling gradient	Limiting gradient	Experimental gradient
Plain or rolling	3.3 per cent (1 in 30)	5 per cent (1 in 20)	6.7 per cent (1 in 15)
Mountainous terrain, and steep terrain having elevation more than 3,000 m above the mean sea level	5 per cent (1 in 20)	6 per cent (1 in 16.7)	7 per cent (1 in 14.3)
Steep terrain upto 3,000 m height above mean sea level	6 per cent (1 in 16.7)	7 per cent (1 in 14.3)	8 per cent (1 in 12.5)

Suppose the road is with zero gradient passing through level land and open side drains are provided with a gradient of say 1 in 300. It may then be necessary to deepen the downstream end of the drain by about 3.3 m for one kilometer length of road. This course is not possible from practical considerations. Hence it is desirable to have a certain *minimum gradient* on roads from drainage point of view, provided topography favours this. The minimum gradient would depend on rain fall run off, type of soil, topography and other site conditions.

A minimum gradient of about 1 in 500 may be sufficient to drain water in concrete drains or gutter; but on inferior surfaces of drains a slope of 1 in 200 or 0.5 percent may be needed where as on kutcha open drains (soil drains) steeper slopes upto 1 in 100 or 1.0 percent may be needed.

Compensation in Gradient on Horizontal Curves

At horizontal curves, due to the turning angle α of vehicles, the curve resistance developed is equal to $T(1 - \cos\alpha)$, as explained in Art. 4.4.9. When there is a horizontal curve in addition to the gradient, there will be increased resistance to fraction due to both gradient and curve. It is necessary that in such cases the total resistance due to grade and curve should not normally exceed the resistance due to the maximum value of the ruling gradient specified. For design purpose, this maximum value may be taken as the ruling gradient and in some special cases as limiting gradient for the terrain. When sharp horizontal curve is to be introduced on a road which has already the maximum permissible gradient, then the gradient should be decreased to compensate for the loss of tractive effort due to the curve.

This reduction in gradient at the horizontal curve is called grade compensation, which is intended to off-set the extra tractive effort involved at the curve. This, is calculated from the relation :

Grade compensation, percent = $\dfrac{30+R}{R}$, subject to a maximum value of 75/R, where R is the radius of the circular curve in metre.

According to the IRC the grade compensation is not necessary for gradients flatter than 4.0 percent and therefore when applying grade compensation correction, the gradients need not be eased beyond 4 percent.

Example 4.21

While aligning a hill road with a ruling gradient of 6 percent, a horizontal curve of radius 60 m is encountered. Find the compensated gradient at the curve.

Solution

Ruling gradient = 6.0%

Grade compensation = $\dfrac{30+R}{R} = \dfrac{30+60}{60} = 1.5\%$

Maximum limit of grade compensation = 75/R = 75/60 = 1.25%

Therefore, compensated gradient = 6.0 − 1.25 = 4.75%

4.5.3 Vertical Curves

Due to changes in grade in the vertical alignment of highway, it is necessary to introduce vertical curve at the intersections of different grades to smoothen out the vertical profile and thus ease off the changes in gradients for the fast moving vehicles.

The vertical curves used in highway may be classified into two categories :

(a) *Summit* curves or *crest* curves with convexity upwards.

(b) *Valley* or *sag* curves with concavity upwards.

Summit curves

Summit curves with convexity upwards are formed in any one of the case illustrated in Fig. 4.35. The deviation angle between the two interacting gradients is equal to the algebraic difference between them. Of all the cases, the deviation angle will be maximum when an ascending gradient meets with a descending gradient i.e., $N = n_1 - (-n_2) = (n_1 + n_2)$.

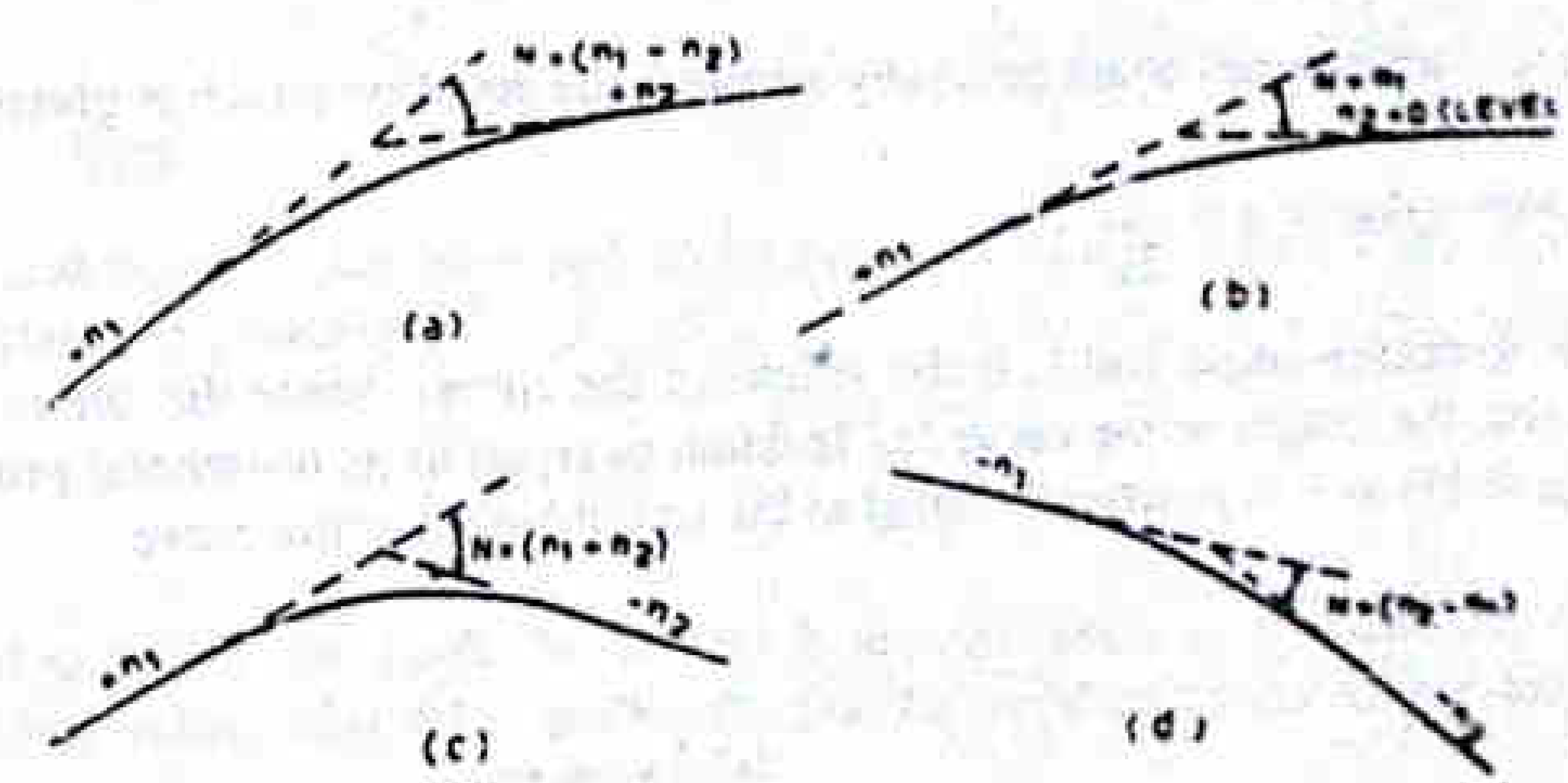


Fig. 4.35 Summit Curves

When a fast moving vehicle travels along a summit curve, the centrifugal force will act upwards, against gravity and hence a part of the pressure on the tyres and spring of the vehicle suspensions is relieved. So there is no problem of discomfort to passengers on summit curves, particularly because the deviation angles on roads are quite small and if the summit curve is designed to have adequate sight distance, the length of the curve would be long enough to ease the shock due to change in gradients.

The only problem in designing summit curves is to provide adequate sight distances. The stopping sight distance or the absolute minimum sight distance should invariably be provided at all sections of the road system and so also on summit curves. As far as possible safe overtaking sight distance or at least intermediate sight distance, equal to twice the stopping sight distance should also be available on these curves for important highways, except when restrictions to overtaking have been strictly imposed at the sections concerned.

Types of summit curves

As the design of summit curves (except low summit or *humps* which do not interfere with sight distance) are governed only by considerations of sight distance, transition curves are not necessary. Circular summit curve is ideal as the sight distance available throughout the length of circular curve is constant. From this view point, transition curve may be said to be even undesirable to be used on summit curves. This is because the radius of curvature and hence the sight distance would vary from point to point along the length of curve. The deviation angles in vertical curves of highways are very small and so between the same tangent points, a simple parabola is nearly congruent with a circular arc; also a parabola is very easy for arithmetical manipulation for computing ordinates. The use of simple parabola as summit curve is found to give good riding comfort too. Because of these reasons in actual practice a simple parabolic curve is used as summit curve instead of the circular arc.

There may be situations where the problem of sight distance does not arise as in the case when the road goes over a *hump*. This may be due to the presence of a culvert, the top of which is above the general level of the road by less than about a metre thus causing a sharp but relatively small summit or hump on the road profile. In such a case the problem is of discomfort to the passengers and so the most suitable curve would be transition curves on either side of this hump. For proper design of humps, the vertical profile should consist of two transition curves on either side of the hump with a level strip in between.

Length of summit curve

Parabolic summit curves are generally adopted, the equation which is given by :

$y = a\,x^2$, with value of $a = \dfrac{N}{2L}$

Here N is deviation angle and L is the length of the curve. Since the summit curve are long and flat, the length of the curve 'L' is taken as equal to its horizontal projection AH (Refer Fig. 4.36) as it is practically equal to the actual length of the curve.

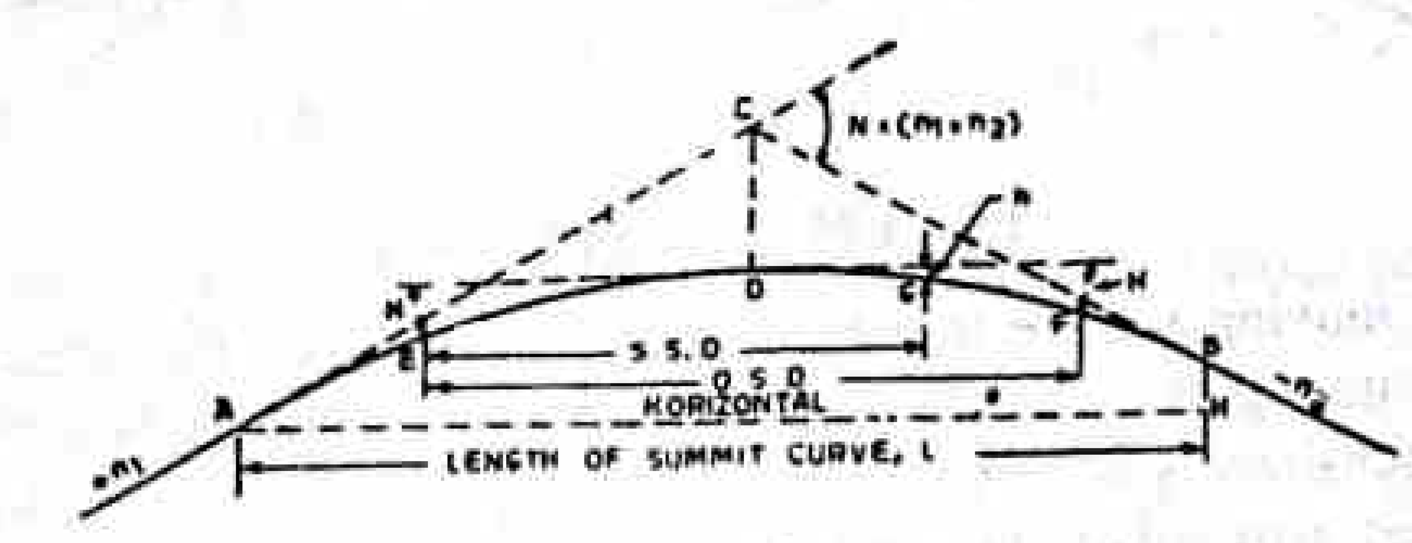


Fig. 4.36 Length of Summit Curve

While designing the length of the parabolic summit curves, it is necessary to consider the stopping sight distance (SSD) and overtaking sight distance (OSD) separately. As mentioned earlier, it is essential to provide sight distance at least equal to the stopping sight distance at all points on a highway so as to avoid accidents due to inadequate sight distance.

Length of summit curve for stopping sight distance (SSD)

Two cases are to be considered in deciding the length

(i) When the length of the curve is greater than the sight distance (L > SSD)

(ii) When the length of the curve is less than the sight distance (L < SSD)

(i) When L > SSD

The general equation for length L of the parabolic curve is given by :

$$L = \frac{NS^2}{(\sqrt{2H}+\sqrt{2h})^2} \tag{4.29}$$

Here L = length of summit curve, m

S = stopping sight distance, (SSD), m

N = deviation angle, equal to algebraic difference in grades, radians or tangent of the deviation angle.

H = height of eye level of driver above roadway surface, m

h = height of subject above the pavement surface, m

The value of H, the height of driver's eye above roadway surface is taken as 1.2 m in India as discussed in Art. 4.3.2. The height of object 'h' above the pavement surface for the purpose of safe stopping distance is taken as 0.15 m as per the IRC standard. Substituting these values in equation 4.29 the length of summit curve is obtained as :

i.e. $$L = \frac{NS^2}{4.4} \tag{4.30}$$

(ii) When L < SSD

The general equation for the length of the parabolic summit curve, when it is less than the sight distance is given by :

$$L = 2S - \frac{(\sqrt{2H}+\sqrt{2h})^2}{N} \tag{4.31}$$

Here the description for L, S, N, H and h are the same as in equation 4.29. By substituting the values of H = 1.2 and h = 0.15 m the length of the curve is obtained as :

$$L = 2S - \frac{4.4}{N} \tag{4.32}$$

Thus to design the length of vertical summit curve providing, for stopping or absolute minimum sight distance, first the safe stopping sight distance is found either by calculation as given in Art. 4.3.2, and equation 4.2 or from the recommended values given in Table 4.5. By using any of the two appropriate formulae by trial (4.30 and 4.32) the required length of summit curve is then calculated. The minimum radius of the parabolic summit curve may be calculated from the relation R = L/N.

Length of summit curve for safe overtaking sight distance (OSD) or Intermediate Sight Distance (ISD)

Two cases to be considered in deciding the length are :

(i) When the length of curve is greater than the overtaking or Intermediate sight distance (L > S) and

(ii) When the length of the curve is less than the overtaking or Intermediate sight distance (L < S)

(i) When L > S

The same general Eq. 4.29 is applicable in this case also. But in this case, the values of H and 'h' both are taken equal to 1.2 m. Substituting h = H in the Eq. 4.29 and simplifying.

$$L = \frac{NS^2}{8H}$$

As discussed in Art. 4.3.3 the height of the eye level of the driver as well as the height of the approaching object are taken as 1.2 m. Substituting the value of H, the height of eye level of driver above the pavement surface equal to 1.2 m.

$$L = \frac{NS^2}{9.6} \qquad (4.33)$$

Here, L = the length of parabolic summit curve,

N = deviation angle, radians or tangent of the deviation angle,

S = overtaking or intermediate sight distance, (OSD/ISD)

(ii) When L < S

The same general equation 4.31 may be used. By substituting H = h and simplifying when L is less than OSD/ISD :

$$L = 2S - \frac{8H}{N}$$

Here again substituting the value of H as 1.2 m, the equation reduces to :

$$L = 2S - \frac{9.6}{N} \qquad (4.34)$$

Here L, S and N are as in Eq. 4.33.

Thus to design the length of vertical summit curve providing for safe or cautious overtaking, the value of the overtaking or intermediate sight distance is calculated from the available data as given in Art. 4.3.3 and Eq. 4.6 or from the recommended value given in Table 4.7 or twice the SSD value as given in Table 4.5, for finding the value of ISD. By using any of the two appropriate formula 4.33 and 4.34, the required length of summit curve is then calculated. The minimum radius of the curve R is calculated from the relation R = L/N as before.

When the deviation angle is small, the length of summit curve generally works out less than the sight distance. In very small deviation angles, the length required some times works out as a negative value indicating that there is no problem of sight restriction at the summit curve.

The minimum lengths of vertical curve for different speeds and for the maximum grade change values (in percent) which do not require vertical curve, as per the ... Table 4.14.

Table 4.14 Minimum Length of Vertical Curves

Design speed (kmph)	Maximum grade change (percent) not requiring a vertical curve	Minimum length of vertical curve, m (for higher grade change values)
35	1.5	15
40	1.2	20
50	1.0	30
65	0.8	40
80	0.6	50
100	0.5	60

The highest point on the summit curve is a distance L n_1/N from the tangent point on the first grade n_1. This is obtained by assuming the summit curve to be a simple parabola of equation : $y = a x^2$ and by differentiating the height y with respect to distance x and equating to zero for the highest point. The summit vertical curve may be plotted by finding the ordinate value Y_1, Y_2, Y_3 ... for different lengths X_1, X_2, X_3 .... from the tangent point, by taking measurements along the tangent lengths.

Valley curves

Valley curves or sag curves are formed in any one of the cases illustrated in Fig. 4.37. In all the cases the maximum possible deviation angle is obtained when a descending gradient meets with an ascending gradient.

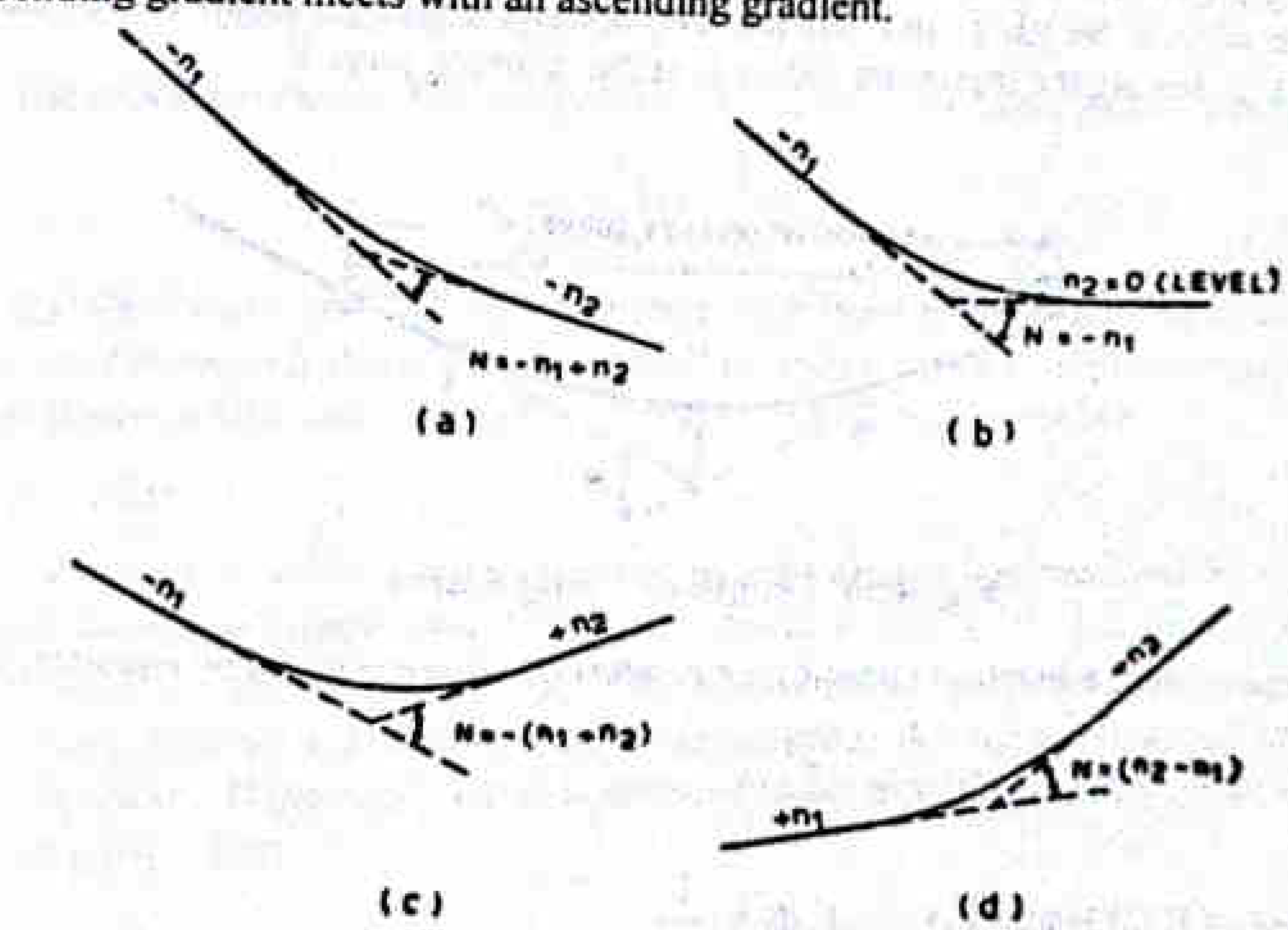


Fig. 4.37 Valley Curves

There is no problem of restriction to sight distance in valley curves during day light. However, during night driving under head lights of vehicles, the sight distance available at valley curve is decreased. The most important factors considered in valley curve design are, (i) impact-free movement of vehicles at design speed or the comfort to the passengers and (ii) availability of stopping sight distance under head lights of vehicles for night driving. The lowest point in the valley curve may be located from considerations of cross drainage.

At the valley curve, the centrifugal force acts downwards adding to the pressure on the springs and the suspensions of the vehicle in addition to that due to weight of the vehicle. Hence the allowable rate of change of centrifugal acceleration should govern the design of the valley curves. Obviously the best shape of valley curve is a transition curve for gradually introducing and increasing the centrifugal force acting downwards. Cubic parabola is generally preferred in vertical valley curves. As the deviation angles are small, the path traversed by the three types of transition curves, viz. spiral, lemniscate and cubic parabola are almost the same as explained in Art. 4.4.7.

During night driving the visibility ahead is dependent on the head light of the vehicles, when the road lighting is not adequate or has not been provided. There is restriction of the sight distance at valley curves as the head light gets intercepted though the beam of light may be slightly inclined upwards. Therefore the *head light sight distance* available at valley curves should be atleast equal to the stopping sight distance. However, there is no problem of overtaking sight distance at valley curves during night as other vehicles with head lights can be seen from a considerable distance.

Length of valley curve

The length of valley transition curve is designed based on the two criteria : (i) the allowable rate of change of centrifugal acceleration of 0.06 m/sec^3 and (ii) the head light sight distance, and the higher of the two values is adopted. Usually the second criterion of head light sight distance is higher and therefore governs the design.

The valley curve is made fully transitional by providing two similar transition curves of equal length (without a circular curve in between). Refer Fig. 4.38, where ABC is the valley curve of total length L and AB and BC are two equal transition curves each of length $L_s = L/2$, having the minimum radius R at the common point B.

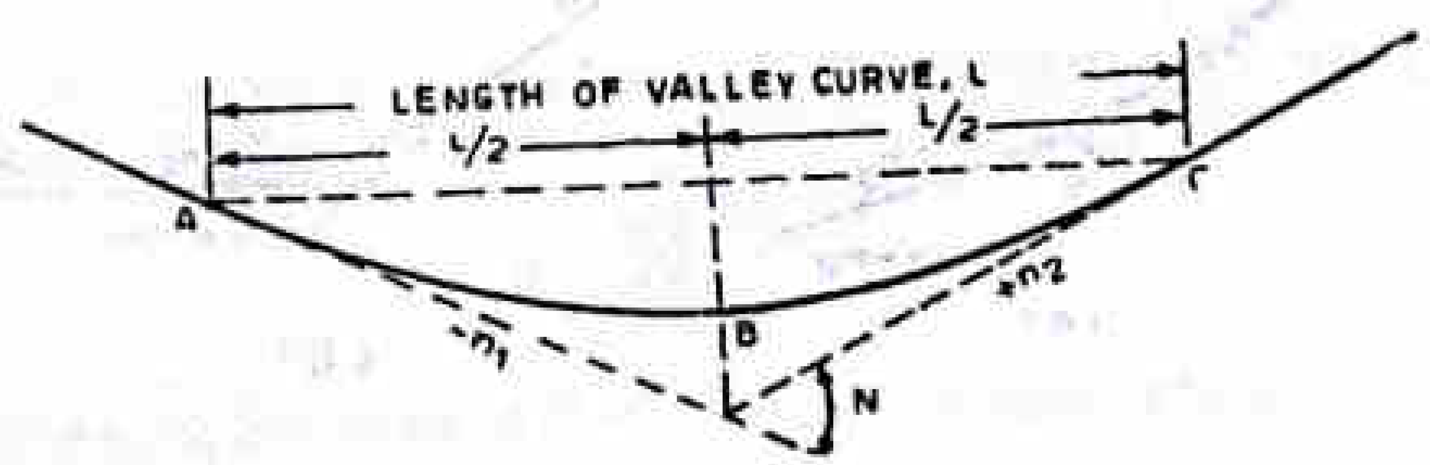


Fig. 4.38 Length of Valley Curve

(1) *The length of transition curve L_s for comfort condition* is given by equation 4.22.

$$L_s = \frac{v^3}{CR}$$

$$\text{Value of R (at length } L_s) = L_s/N = \frac{L}{2N}$$

$$L_s = \frac{v^3}{C.L_s} \times N \text{ or } L_s^2 = \frac{Nv^3}{C}$$

$$L_s = \left[\frac{Nv^3}{C}\right]^{\frac{1}{2}}$$

$$L = 2\,L_s = 2\left[\frac{Nv^3}{C}\right]^{\frac{1}{2}} \qquad (4.35)$$

where N is the deviation angle, v is speed in m/sec and C is the allowable rate of change of centrifugal acceleration which may be taken as 0.6 m/sec^3.

$$V \text{ kmph} = \frac{V}{3.6} \text{ m/sec}$$

$$L_s^2 = \frac{NV^3}{0.6 \times 3.6^3}$$

$$L_s = 0.19\,(NV^3)^{1/2}$$

$$L = 2\,L_s = 0.38\,(NV^3)^{1/2} \qquad (4.36)$$

Hence the total length of valley curve is given by equation :

$$L = 2\left[\frac{Nv^3}{C}\right]^{\frac{1}{2}} = 0.38\,(NV^3)^{1/2}$$

where L = total length of valley curve,

N = deviation angle in radius or tangent of the deviation angle or the algebraic difference in grades.

V = design speed, kmph

The minimum radius (R metre) of the valley curve for cubic parabola is given by :

$$R = \frac{L_s}{N} = \frac{L}{2N} \qquad (4.37)$$

(2) *The length of valley curve for head light sight distance* may be determined for the two conditions : (i) when the total length of valley curve L is greater than the stopping sight distance SSD and (ii) when L is less than SSD, as given below.

(i) L > SSD

The length of valley curve L is assumed to be greater than the head light sight distance which should be atleast equal to SSD, as shown in Fig. 4.39. Let the height of the head light be h_1 and the focused portion of the beam of light be inclined at an angle α upwards. The sight distance available will be minimum when the vehicle is at the lowest point on the sag curve. If the valley curve is assumed to be of parabolic shape, with equation $y = a x^2$, where $a = N/2L$.

$$h_1 + S \tan\alpha = a\,S^2 = \frac{N\,S^2}{2L}$$

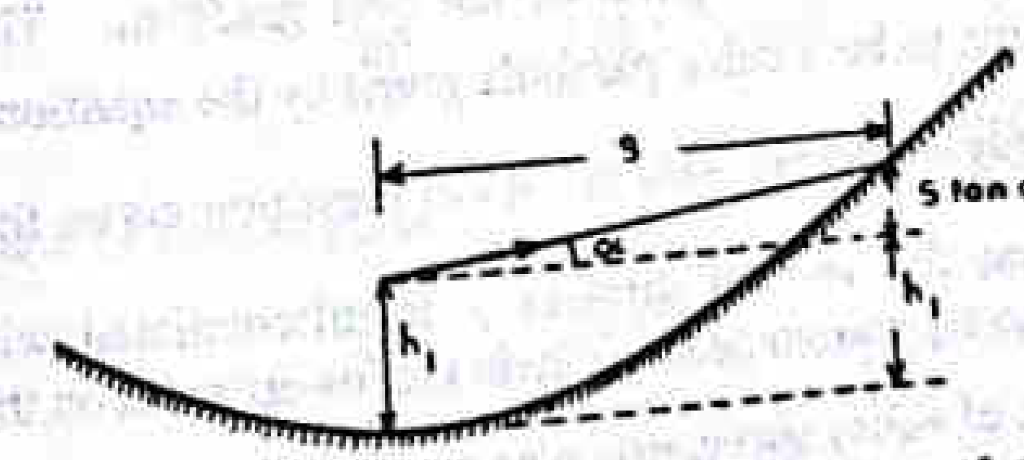


Fig. 4.39 Head Light Sight Distance when L > S

$$L = \frac{N S^2}{(2h_1 + 2 S \tan \alpha)}$$

If the average height of the head light is taken as $h_1 = 0.75$ m and the beam angle $\alpha = 1°$, by substituting these in the above equation,

$$L = \frac{N S^2}{(1.5 + 0.035 S)} \tag{4.38}$$

where, L = total length of valley curve, m ($L > S$)

S = SSD, m

N = deviation angle = $(n_1 + n_2)$, with slopes $- n_1$ and $+ n_2$

(ii) L < SSD

Refer Fig. 4.40. Let the vehicle be at the start of the valley curve or at the tangent point TP, for minimum sight distance. Therefore,

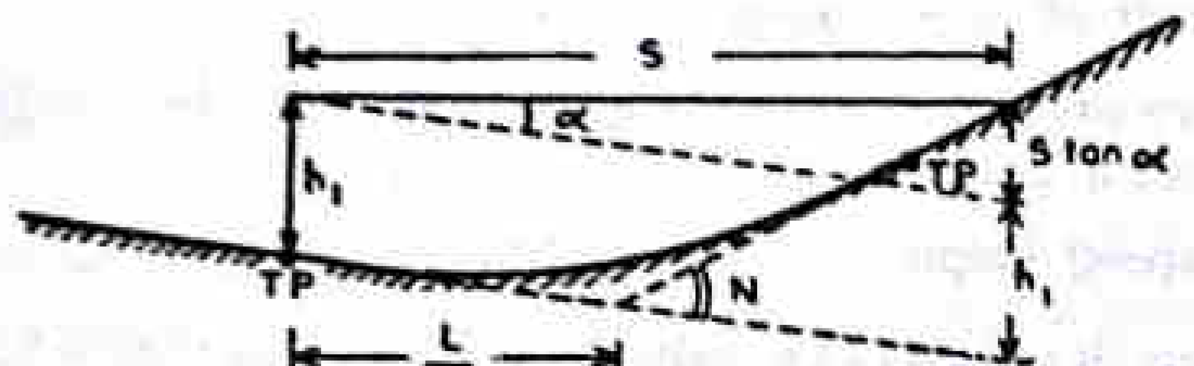


Fig. 4.40 Head Light Sight Distance when L < S

$$h_1 + S \tan \alpha = \left(S - \frac{L}{2}\right) N$$

$$L = 2S - \frac{(2h_1 + 2 S \tan \alpha)}{N}$$

Substituting $h_1 = 0.75$ m and $\alpha = 1°$, when $L < S$

$$L = 2S - \frac{(1.5 + 0.035 S)}{N} \tag{4.39}$$

The lowest point on the valley curve is to be located for providing the cross drainage facility. The lowest point on the valley curve will be on the bisector of the angle between the grades, if the gradients on either side are equal. When the gradients are not equal, the lowest point lies on the side of flatter grade, and this point is at a distance $X_0 = L\sqrt{n_1/2N}$ from the tangent point of the first grade n_1. This is obtained by assuming the valley curve to be a cubic parabola given by the equation; $y = bx^3$, where $b = \frac{2N}{3L^2}$. This practically coincides with the spiral transition curve for small deflection angles of valley curves; the vertical distance y is differentiated with respect to the horizontal distance x and equated to zero to obtain the lowest point on the curve.

The minimum length of valley curve, may also be obtained from Table 4.14.

Example 4.22

A vertical summit curve is formed at the intersection of two gradients, + 3.0 and − 5.0 percent. Design the length of summit curve to provide a stopping sight distance for a design speed of 80 kmph. Assume other data

Solution

(i) $$SSD = 0.278\, V t + \frac{V^2}{254 f}$$

As there is ascending gradient on one side of the summit and descending gradient on the other side, the effect of gradients on the SSD is assumed to get compensated and hence ignored in the calculations.

Assuming $t = 2.5$ sec and $f = 0.35$ for $V = 80$ kmph,

$$SSD = 0.278 \times 80 \times 2.5 + \frac{80^2}{254 \times 0.35}$$

$$= 55.6 + 72.0 = 127.6, \text{ say } 128 \text{ m}$$

(ii) Deviation angle $N = 0.03 - (-0.05) = 0.08$

Assuming L > SSD (Eq. 4.30),

$$L = \frac{N S^2}{4.4} = \frac{0.08 \times 128^2}{4.4}$$

$$= 297.9 \text{ m } (> 128 \text{ m})$$

Therefore length of summit curve = 298 m

This value is higher than the minimum specified length of 50 m at 80 kmph speed as per Table 4.14.

Example 4.23

An ascending gradient of 1 in 100 meets a descending gradient of 1 in 120. A summit curve is to be designed for a speed of 80 kmph so as to have an overtaking sight distance of 470 m.

Solution

$$n_1 = +\frac{1}{100}, n_2 = -\frac{1}{120}$$

$$N = \frac{1}{100} - \left(-\frac{1}{120}\right) = \frac{11}{600}$$

If L > OSD

From Eq. 4.33 length of summit curve,

$$L = \frac{N S^2}{9.6} = \frac{11 \times 470^2}{600 \times 9.6} = 422 \text{ m}$$

As this value is less than OSD of 470 m, assume L less than OSD.

If L > OSD

From equation 4.34 length of summit curve,

$$L = 2S - \frac{9.6}{N}$$

$$= 2 \times 470 - \frac{9.6 \times 600}{11} = 416.4 \text{ m, say } 417 \text{ m}$$

This value is less than 440 m.

Therefore, the length of summit curve = 417 m.

Example 4.24

A vertical summit curve is to be designed when two grades, + 1/50 and – 1/80 meet on a highway. The stopping sight distance and overtaking sight distance required are 180 and 640 m respectively. But due to site conditions the length of vertical curve has to be restricted to a maximum value of 500 m if possible. Calculate the length of summit curve needed to fulfil the requirements of (a) Stopping sight distance (b) Overtaking sight distance or atleast Intermediate sight distance and discuss the results.

Solution

$$N = +\frac{1}{50} - \left(-\frac{1}{80}\right) = \frac{13}{400}$$

(a) Requirements of stopping sight distance

$$SSD = 180 \text{ m}$$

Assume L > SSD

$$L = \frac{NS^2}{4.4} = \frac{13 \times 180}{400 \times 4.4} = 239.3 \text{ m}$$

As this length is greater than SSD the assumption is correct.

The length of summit curve required is 240 m which is less than the prescribed maximum limit of 500 m.

(b) Requirement of overtaking sight distance

$$OSD = 400 \text{ m}$$

Assume L > OSD

$$L = \frac{NS^2}{9.6} = \frac{13 \times 640^2}{400 \times 9.6} = 1387 \text{ m}$$

As the length of summit curve obtained is higher than the sight distance, length required is 1387 m.

As suggested in problem if the length of the summit curve is restricted to a value less than 500 m, it is not possible to provide the required O.S.D. of 640 m.

Therefore, to provide limited opportunities for overtaking, Intermediate Sight Distance (ISD) equal to twice the SSD of 180 × 2 = 360 m may be provided if possible.

If L > SD

$$L = \frac{NS^2}{9.6} = \frac{13 \times 360^2}{400 \times 9.6} = 439 \text{ m}$$

As this value is greater than the SD of 360 m, the assumption is correct. It is possible to provide the ISD of 439 m to allow limited overtaking operations and the length of summit curve in this case is less than the maximum available length of 500 m.

Example 4.25

A valley curve is formed by a descending grade of 1 in 25 meeting an ascending grade of 1 in 30. Design the length of valley curve to fulfil both comfort condition and head light sight distance requirements for a design speed of 80 kmph. Assume allowable rate of change of centrifugal acceleration C = 0.6 m/sec^3.

Solution

$$N = -\frac{1}{25} - \frac{1}{30} = -\frac{11}{150}$$

$$V = 80 \text{ kmph, } v = 80/3.6 = 22.2 \text{ m/sec}$$

(i) Comfort Condition

From Eq. 4.34,

$$L = 2\left[\frac{Nv^3}{C}\right]^{\frac{1}{2}} = 2\left[\frac{11}{150} \times \frac{22.2^3}{0.6}\right]^{\frac{1}{2}} = 73.1 \text{ m}$$

(ii) Head Light Sight Distance Condition

Neglecting the ascending and descending gradients at the valley curve using Eq. 4.1 and assuming t = 2.5 secs. and f = 0.35, SSD = v t + $v^2/2gf$ = 22.2 × 2.5 + ($22.2^2/2$ × 9.8 × 0.35) = 127.3 m

If L > SSD, using Eq. 4.38

$$L = \frac{NS^2}{(1.5 + 0.035S)} = \frac{11 \times 127.3^2}{150(1.5 + 0.035 \times 127.3)} = 199.5 \text{ m}$$

As this value is higher than the SSD of 127.3 m, the assumption is correct. The valley curve length based on head light sight distance being higher than that based on comfort condition, the design length of valley curve is 199.5 or say, 200 m.

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PROBLEMS

1 What are the objects of highway geometric design ? List the various geometric elements to be considered in highway design.

2 Explain the role of pavement surface characteristics in highway geometric design. State the factors affecting friction between pavements and tyres of vehicles ?

3 Explain camber. What are the objects of camber ? Discuss the factors on which the amount of camber to be provided depends. Specify the recommended ranges of camber for different types of pavement surfaces.

4 Discuss the effects of shape of camber and the effects of providing steep cross fall.

5 Enumerate the factors governing the width of carriage way. State the IRC specifications for width of carriage way for various classes of roads.

6 Write short notes on :

(a) Traffic separators

(b) Kerbs

(c) Roads margins

(d) Pavement unevenness

(e) Shoulders

(f) Width of formation (or road way)

(g) Right of way.

7 Draw the typical cross sections of the following roads indicating the width of pavement, roadway and land

(a) M.D.R. in embankment

(b) O.D.R. in cutting

(c) National highway in embankment in rural area

(d) National highway in cutting

(e) A city road

(f) A divided highway in urban area

8 Explain sight distance and factors causing restrictions to sight distance. Explain the significance of stopping, intermediate and overtaking sight distances.

9 What are the factors on which the stopping sight distance depends ? Explain briefly ?

10 Explain total reaction time of driver and the factors on which it depends. Explain "PIEV" theory.

11 Derive an expression for finding the stopping sight distance at level and at grades.

12 Calculate the stopping sight distance for a design speed of 100 kmph. Take the total reaction time 2.5 second and the coefficient of friction = 0.35.

13 Find the stopping sight distance for a design speed of 65 kmph. Assume suitable data. What are sight distance requirements at a gradient of 1 in 40.

(*Hint for Solution* : Assume total reaction time as 2.5 second and design coefficient of friction as 0.36; find the stopping sight distances for level, ascending and descending grades).

14 State factors on which the overtaking sight distance depends. Explain briefly.

15 Derive an expression for calculating the overtaking sight distance on a highway.

16 Why are overtaking zones provided ? What is the basis of deciding its length ? Draw a neat sketch and show the signs to be installed and their positions.

17 The speeds of overtaking and overtaken vehicle are 80 and 60 kmph respectively. If the acceleration of the overtaking vehicle is 2.5 kmph per second, calculate the safe passing sight distance for

(a) one-way traffic

(b) two-way traffic

18 In problem 17, what should be the length of overtaking zone. Show the position of signs by means of a sketch.

19 Find the safe overtaking sight distance for a highway having a design speed of 100 kmph. Assume all data suitably.

[*Hint for Solution* : Assume speed of overtaken vehicle = (100 − 16) kmph; maximum acceleration of overtaking vehicle = 1.6 kmph/sec, as given in Table 4.6].

20 Discuss the factors to be considered in deciding the sight distance at intersections.

21 (a) Enumerate the design elements to be considered at the horizontal alignment.

(b) What are the effects of speed on horizontal alignment design ? What are design speeds for different classes of roads specified by the IRC ?

22 (a) Explain superelevation. What are the factors on which the design of superelevation depends ?

(b) Explain maximum and minimum superelevations.

23 Derive an equation for finding the superelevation required if the design coefficient of lateral friction is 'f'.

24 Enumerate the steps for practical design of superelevation ?

25 Design the superelevation required at a horizontal curve of radius 300 m for speed of 60 kmph. Assume suitable data.

26 Calculate the maximum allowable speed on a horizontal curve of radius 350 m if the maximum allowable values of lateral coefficient of friction is 0.15 and rate of superelevation is 0.07.

27 A radius of 250 m has to be provided at a locality due to site restrictions in a National Highway with design speed 100 kmph. Design the superelevation. Should there be restriction in speed ?

28 Explain with the aid of neat sketches the methods of eliminating camber and introduction of superelevation.

29 What is the basis of calculating absolute minimum radius of horizontal curve ? What do you understand by ruling minimum radius of curve ?

30 Calculate the absolute minimum and ruling minimum radius of horizontal curve for a design speed of 80 kmph.

31 State the objects of widening pavement on horizontal curves ? What are the factors on which the design of widening depends ? Explain.

32 Derive an expression for finding the extra widening required on horizontal curve.

33 (a) Why should the psychological widening be added to the mechanical widening ?

(b) How is the widening of pavements introduced in the field ?

(c) What is off-tracking ? Explain with sketches.

(d) Determine the off-tracking of a vehicle with wheel base 7.0 m while negotiating a horizontal curve of radius 100 m.

34 Calculating the extra width of pavement required on a horizontal curve of radius 700 m on a two lane highway, the design speed being 80 kmph. Assume wheel base $l = 6$ m.

35 In a mountainous terrain, a circular curve of radius 50 m and length 40 m has transition of 20 m on both ends. Calculate extra widening if the design speed is 30 kmph. Suggest suitable method of providing the widening on a two lane pavement.

36 What are the objects of providing transition curves on the horizontal alignment highways ? Explain.

37 List the various types of transition curves used in highways. What is an ideal transition curve ? Explain.

38 Discuss the factors to be considered while designing the length of transition curve.

39 Derive an expression for finding length of transition curve on horizontal alignment of highways.

40 The radius of a horizontal curve is 400 m, the total pavement width at curve is 7.6 m and the superelevation is 0.07. Design the transition curve length for a speed of 100 kmph. Assume pavement to be rotated about the inner edge.

41 Calculate the length of transition curve for a design speed of 80 kmph at horizontal curve of radius 300 m in a rural area. Assume suitable data.

(*Hint for Solution* : Solve the problem as in Example 4.16).

42 (a) Explain curve resistance and compensation in gradient on horizontal curves.

(b) There is a horizontal curve of radius 60 m on a stretch of hill road with a gradient of 5.0%. Determine the grade compensation.

43 Explain with sketches how sight distance is restricted on horizontal curves and how the desired sight distance could be obtained.

44 The stopping sight distance required for a highway is 80 m. Find the required set back distance from centre line of a circular curve of radius 300 m assuming the length of the curve is greater than the sight distance.

45 The overtaking sight distance required on a highway is 250 m. Find the required clearance of obstruction from centre line of a circular curve of radius 350 m and length 180 m. Assume two lane highway with d = 1.9 m.

46 A national highway passing through a flat terrain has a horizontal curve of radius equal to the ruling minimum radius. If the design speed is 100 kmph, calculate absolute minimum sight distance, superelevation, extra widening and length of transition curve. Assume necessary data suitably.

47 Enumerate the various design factors controlling the vertical alignment of highways.

48 (a) Explain ruling, maximum and exceptional gradients. Specify the values recommended by IRC for plains and hill.

(b) State the various considerations in deciding the ruling gradient of highway.

49 The ruling gradient of a hill road is 1 in 20. What should be the compensation in gradient and compensated gradient on a horizontal curve of radius 80 m after allowing for curve resistance ?

50 Explain summit and valley curves and the various cases when these are formed while two different gradients meet.