CHAPTER 3

Geometric Design of Highway

INTRODUCTION

This unit contents the principle of road layouts. A highway has many visible dimensions and the design of visible dimensions is known as Geometric Design. These visible dimensions are:

(*i*) Horizontal and vertical alignment
(*ii*) Sight distance considerations
(*iii*) Intersection elements
(*iv*) Cross sectional components

Geometric of highway should be designed to provide efficient and comfortable operation of traffic. Layout design of road sections joining two roads with different directions is referred to as geometric design of horizontal curves and the joining of two roads with different gradients is referred to as geometric design of vertical curves. The design of road sections for the purpose of proper delineation of vehicular paths is referred to as channelization design.

3.1 Factors Controlling Geometric Design

There are certain basic design controls and criteria which govern the geometric features of a highway.

(*i*) Topography
(*ii*) Design speed
(*iii*) Road user characteristics
(*iv*) Vehicle characteristics
(*v*) Traffic (its volume, directional distribution and composition including the future estimates)
(*vi*) Traffic capacity
(*vii*) Environmental considerations
(*viii*) Economy in construction, maintenance and operation of vehicles

NOTE: Design speed of a highway is important for economic operation and has a great bearing on the safety of the highway. It plays a vital role in determining the geometric design.

3.1.1 Topography or Terrain

The topography of the area from which the road is passing also controls the geometric design of highway. The classification of topography is based on the cross slope of the country as Plain, Rolling, Mountainous and Steep.

Table-3.1 : Terrain classification on the basis of cross slope

Type of Terrain	Plain	Rolling	Hilly	Steep
Cross-slope of country(in %)	0 - 10	10 - 25	25 - 60	> 60

An extensive survey of 42,000 km of roads in India as part of the road user cost study has enabled the development of the following systems of terrain classification.

Table-3.2 : Terrain classification on the basis of rise and fall, and curvature

S.No.	Terrain classification	Rise and fall (m/km)	Curvature (degree/km)
1.	Plain: (a) Low curvature (b) High curvature	0 - 15 0 - 15	0 - 50 > 51
2.	Rolling: (a) Low curvature (b) High curvature	16 - 30 16 - 30	0 - 100 > 100
3.	Hilly: (a) Low curvature (b) High curvature	> 31 > 31	0 - 200 > 201

The quantification of curvature and rise and fall is given as

$$\text{Average curvature of section } AB = \frac{\phi_1 + \phi_2 + \phi_3 + \ldots\ldots\ldots\ldots + \phi_n}{\text{Distance } AB(\text{km})}$$

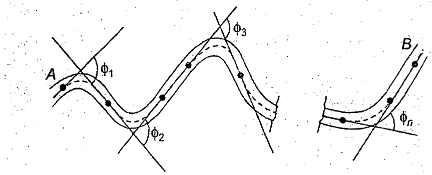


Figure-3.1 : Plan of section

$$\text{Average rise of section } AB = \frac{h_1 + h_3 + \ldots\ldots\ldots + h_m}{\text{Distance } AB}$$

$$\text{Average fall of section } AB = \frac{h_2 + h_4 + \ldots\ldots\ldots + h_n}{\text{Distance } AB}$$

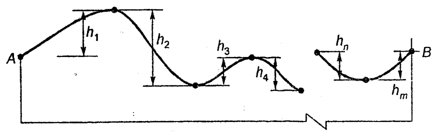


Figure-3.2 : Longitudinal profile of section

3.1.2 Design Speed

Design speed is the maximum speed at which vehicles can continuously run safely under the favorable conditions. It is the most important factor controlling the geometric design elements of highway. The design speed obviously has to be correlated with the terrain conditions and the classification of the highway (like National Highway, State Highway etc.) as given in Table 3.3.

Table-3.3 : Suggested design speeds in India for Rural Highways (kmph)

Classification	Plain Terrain		Rolling Terrain		Mountainous Terrain		Steep Terrain	
	Ruling	Minimum	Ruling	Minimum	Ruling	Minimum	Ruling	Minimum
National Highways and State Highways	100	80	80	65	50	40	40	30
Major District Roads	80	65	65	50	40	30	30	0
Other District Roads	65	50	50	40	30	25	25	20
Village Roads	50	40	40	35	25	20	25	20

The pattern of speed distribution on a highway will be discussed in the following unit. Normally " ruling design speed" should be the guiding criteria for correlating the various geometric design features. "Minimum design speed" may however be adopted in sections where site condition, including costs, do not permit a design based on the "ruling design speed".

For limited access facilities like expressways, a higher value is desirable. For urban streets, the design speeds adopted in India according to IRC are given in Table 3.4.

Table-3.4 : Design Speeds for Urban Roads

Classification of Roads	Arterial road	Sub-arterial road	Collector road	Local road
Design Speed (kmph)	80	60	50	30

Do you know? A design speed of 120 kmph has been adopted for the new expressway being constructed between Ahmedabad and Vadodara.

3.1.3 Road User Characteristics

They are generally helpful in the design of sight distance i.e. stopping sight distance and overtaking sight distance. Driver characteristics that influence safety are vision and hearing. A driver takes a certain amount of time to respond to a particular traffic situation. This time is called as reaction time.

The process of applying brakes in any situation is not an instantaneous phenomenon while it is a time consuming phenomenon in which psychological process is involved called as Perception, Intellection, Emotion and Volition (PIEV) as shown in Figure 3.3.

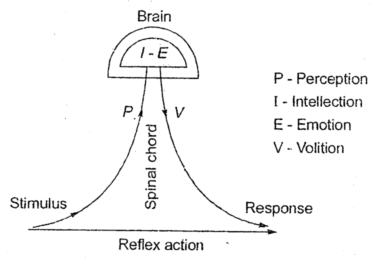


Figure-3.3 : Reaction time and PIEV process

Perception time is the time required for transmission of the sensations received through eyes, ears and body to the brain and the spinal chord by the nervous system. After perception intellection occurs, that is the formation of new thoughts and ideas. It is the time required for understanding the situation.

After these two stages is emotion, based on the situations like fear or anger. Emotion time of a driver is likely to vary considerably depending upon the problems involved after that.

The final decision sent by the brain to the muscle and this actual act of taking a decision to produce action is done through volition time. 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 in complex situations.

Pedestrian characteristics that influence the design of pedestrian facilities are speed and space occupied.

NOTE: A pedestrian speed of 1.2 m/s is commonly taken for design.

3.1.4 Vehicular Characteristic

Vehicle characteristics also play an important role in the geometric design of highway. Vehicle characteristics include width, length, height, weight and axle configuration of a vehicle.

(*i*) The width of a vehicle determines the width of lane.

(*ii*) Wheel base of the vehicle governs the turning path of the vehicle.

(*iii*) The height of a vehicle affects the vertical clearance.

(*iv*) The weight and axle configuration of a vehicle are vital features for the design of pavements.

The maximum dimension of vehicle given by the IRC are:

Width	
1. Motor vehicle other than transport vehicle	2.5 m
2. Transport vehicle	2.7 m
Length	
1. Motor vehicle other than transport having not more that two axles	9.5 m
2. Transport vehicles with rigid frame with two or more axles	11.25 m
3. Articulated vehicle with more than two axles	16 m
4. Truck/trailer or tractor/trailer combinations	18 m
Height	
1. Double decker buses	4.75 m
2. Others for normal application	3.8 m
3. Others for carrying ISO containers	4.2 m

Since there is large variation of vehicle dimensions in mixed traffic, so it is required to specify a certain design vehicle. A design vehicle is a selected motor vehicle, the weight, dimensions and operating characteristics of which are used to establish highway design controls to accommodate vehicles of a designated type.

3.1.5 Design Hourly Volume and Capacity

The traffic 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 highest hourly traffic volume. Therefore a reasonable value of traffic volume is decided for the design and this traffic volume is called the design hourly volume.

Design traffic depends upon the rate of growth of traffic, the design period, importance of road in the system, nature of roadside development etc.

Under mixed traffic conditions, the different types of vehicles need to be converted to a common unit known as "Passenger Car Unit" (PCU) by multiplying their number with relevant equivalency factors. Tentative values of equivalency factors are given in Table 3.5 for rural roads.

Table-3.5 : Equivalency factors for various types of vehicle

S.No.	Vehicle type	Equivalency factor
	Fast Vehicles	
1.	Motor-cycle or Scooter	0.50
2.	Passenger car, Pick-up van or Auto-rickshaw	1.00
3.	Agricultural tractor, light commercial vehicle	1.50
4.	Truck or Bus	3.00
5.	Truck-trailer, Agricultural tractor-trailer	4.50
	Slow Vehicles	
6.	Cycle	0.50
7.	Cycle Rickshaw	2.00
8.	Hand cart	3.00
9.	House drawn vehicle	4.00
10.	Bullock cart*	8.00

*For small bullock carts, a value of 6 will be appropriate

Future Traffic Estimates

The design period used for a flexible pavement generally varies from 15 to 25 years. A period of 20 years is widely used as a basis for design. The future traffic estimates should be calculated to include the following components:

(*i*) Current traffic: Existing and attracted (or diverted)

(*ii*) Future traffic:

(a) Normal traffic growth

(b) Generated traffic

(c) Development traffic

3.2 Basic Consideration for the Design of Highway

(*i*) The design should be safe and efficient for both during day light and at night and also in good and bad weather.

(*ii*) The design should be suitable for the both daily and at peak hours, traffic volume and also for the future anticipated traffic.

(*iii*) The design should confirm to the design speed, and characteristics of vehicles and their drivers using the road. It should take into account not only those vehicles that are at present using the road but for also those that may be expected to use it during its life time.

(*iv*) The design should be consistent i.e. there should not be abrupt changes so that the drivers are not confronted with difficult and serious situations.

(*v*) The designer should have a full knowledge of the conditions under which a vehicle is going to operate. This has a bearing in the design of motor engines, gear box and the length of vehicle in the case of mountainous terrain, etc.

(*vi*) The design should provide an attractive and pleasing view to the road user and those who live along it.

(*vii*) The design must be complete including traffic signs, signals, roadside treatment, etc.

(*viii*) The design should be as simple as possible from the user's as well as from construction point of view. Too many changes in the cross-section or different types of surfaces will create difficulties in construction.

(*ix*) The maintenance cost should be as minimum as possible.

3.3 Cross Sectional Element

These are the features of the cross-section of the pavement that effects the life of the pavement, riding comfort and safety. Following are the cross sectional elements of the pavement:

(*i*) Right of way
(*ii*) Width of formation
(*iii*) Road margins
(*iv*) Medians
(*v*) Kerbs
(*vi*) Width of pavement or carriageway
(*vii*) Camber or cross slope
(*viii*) Pavement characteristics

3.3.1 Right of Way

Right of way is the land acquired to accommodate 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 its future development. The right of way should be adequate to accommodate all the element that make up the cross-section of the highway.

The land width is governed by:

(i) Width of formation: It depends upon type of highway, roadway width and road margins.

(ii) Height of embankment of cutting depth: It is governed by the topography.

(iii) Sight distance: On horizontal curves there is restriction to the visibility on the inner side of the curve due to obstructions like building etc. so at sharp curves it is beneficial to acquire a wide strip of land.

(iv) Drainage system: Size of drains depends upon the rainfall, topography and runoff.

Table-3.6 : Right of way width for different classes of roads in India (in m)

Class of Road	Plain and rolling terrain				Mountainous and steep terrain	
	Rural areas		Urban areas		Rural areas	Urban areas
	Normal	Range	Normal	Range	Normal	Normal
National and State Highways	45	30 - 60	30	30 - 60	24	20
Major District Roads	25	25 - 30	20	15 - 25	18	15
Other District Roads	15	15 - 5	15	15 - 20	15	12
Village Roads	12	12 - 18	10	10 - 15	9	9

In order to prevent ribbon development along highways, it is sometimes necessary to establish "control lines" and "building lines" as shown in Figure 3.4.

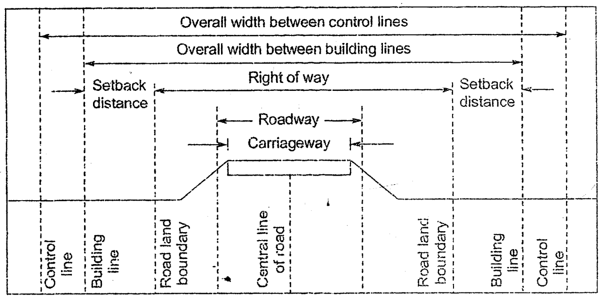


Figure-3.4 : Road margins

Control line is a line which represents the nearest limits of futures uncontrolled building activity in relation to a road. A building line is a line, on either side of the road, between which and the road, no building activity is permitted at all.

Table-3.7 : Standards for building lines and control lines in India

Class of Road	Plain and rolling terrain			Mountainous and steep terrain			
	Rural areas		Urban and industrial area	Distance between building line and road boundary (in m)			
	Width between building lines (in m)	Width between control lines (in m)	Distance between building line and road boundary (in m)	Rural areas		Urban areas	
				Normal	Excep-tional	Normal	Excep-tional
National and State Highways	80	150	3 - 6	5	3	5	3
Major District Roads	50	100	3 - 5	5	3	5	3
Other District Roads	25 - 30	35	3 - 5	5	3	5	3
Village Roads	25	30	3 - 5	5	3	5	3

In urban situations, the space available is generally restricted and costly. Table 3.8 represents current Indian Practice.

Table-3.8 : Space standards for urban roads

Category of Road	Minimum space (in m)
Expressway (6 lane divided)	60
Arterial street (4 lane divided with cycle tracks)	50 - 65
Sub-arterial street (4 lane divided)	30 - 40
Collector street (4 lane)	25 - 35
Local street such as main residential street, shopping street in residential area and minor residential street.	10 - 20

NOTE: Building activity is not totally banned between the building line and the control line, the nature of buildings permitted here is controlled.

3.3.2 Width of Formation

Total width of a road at the level of the bottom of the pavement is termed as width of formation. It is the sum of width of pavements or carriageway including separators if any and shoulders. Roadway width is the total top width of a road. Sometimes, the roadway width itself is known as the formation width.

IRC has suggested the minimum width of carriageway for various class of roads as given in Table 3.9.

Table-3.9 : Width of carriageway for various classes of roads

A - Roads				
Class of Roads	Width of carriageway (in m)			
	Single lane	Two lanes without raised kerbs	Two lanes with raised kerbs	Multi-lane pavements per lane
(a) National and state highways	3.75	7.0	7.5	7.5
(b) Major district roads	3.75	–	–	–
(c) Other district roads	3.75	–	–	–
(d) Village roads	3.75	–	–	–

B - Bridges	
Bridge Road	Clear width of carriageway on road bridges (in m)
(a) Single lane bridges	3.75
(b) Two lane bridges	7.5
(c) Bridges on divided highways with four lane dual carriageway (2 lanes on either side of the central dividing verge)	7.5 for two lane carriageway

Single lane pavements in India are often provided with paved shoulders 0.9 m on either side to facilitate crossing and overtaking manoeuvers. Current Indian practise as regards roadway width is summarised in the Table 3.10.

Table-3.10 : Width of roadway for various class of roads

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

3.3.3 Road Margins

These are the areas which are not used as a regular roadway. The various elements included in the road margins are as shown in Figure 3.5.

Shoulders

These are the portion provided along the carriageway to provide lateral support to the pavement and for giving working space for stopped vehicles. It should be strong enough to bear the weight of a fully loaded truck even in wet conditions. Minimum width of shoulder recommended by IRC is 2.5 m.

Colour of the shoulder should preferably be different from that of the pavement so as to be distinct. It should be rougher than the traffic lanes so that users do not use it as regular traffic lane.

NOTE: (*i*) Shoulder width will be one half of the difference between the roadway width and carriage way width.
(*ii*) Shoulder use increases rapidly with a decrease in pavement width below 7.0 m.

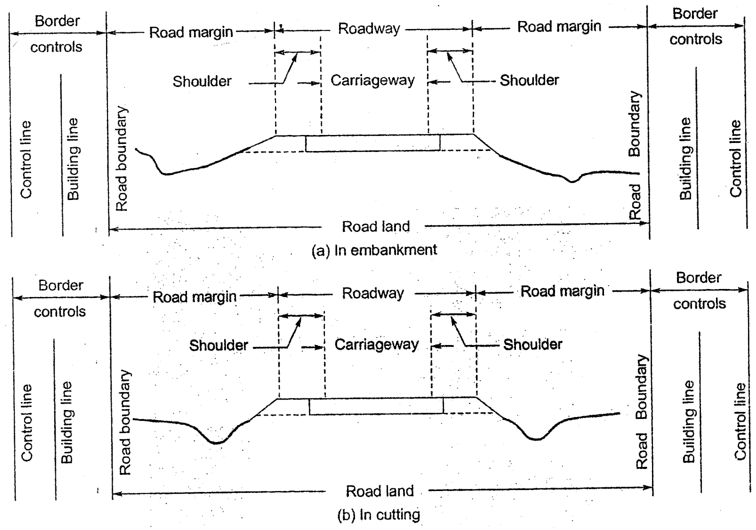


***Figure-3.5 :** Cross section details*

Parking Lane

These are generally provided on urban road to allow kerb parking. Parallel parking is generally provided for safe moving of vehicle. However angle parking is also provided. Parking lane should have sufficient width. For parallel parking 3.0 m width is required.

Lay Bye

These are provided near public conveniences with guide maps to enable drivers to stop clear off the carriageway. Laybye normally have width of 3.0 m and atleast 30 m length.

Cycle Track

These are provided in urban areas where volume of cycle traffic on the road is very high. Minimum width of cycle track is 2 m and it may be increased by 1.0 m for each additional cycle lane.

Drive Way

These are used to connect the highways with commercial establishment like fuel stations, service-stations etc. It should be fairly away from the intersections and have large radius but width should be minimised to reduce the length of cross walks.

Footpath

These are provided in urban roads having heavy vehicular as well as pedestrian traffic, to provide the protection to pedestrians. Minimum width of footpath should be 1.5 m and it may be increased based on the pedestrian traffic volume.

Surface of footpath should be smooth as even smoother than the adjacent traffic lane to keep the pedestrian on the footpath.

Guard Rail

These are provided at the edge of the shoulder when road is constructed on fill to prevent vehicles from running off the embankment. Guard stones are installed at suitable distances to provide better night visibility on the curves under head lights of vehicles.

Service Road

These are provided parallel to the highway and isolated by a separator and access to highway provided only at selected points. They are provided to avoid congestion on the highway and expressways.

3.3.4 Median

A road on which traffic in one direction of travel is separated from that in the opposite direction is called a divided highway and the dividing strip in the middle of the roadway is known as median. IRC recommends a minimum desirable width of 5.0 m for medians of rural highway, which may be reduced to 3.0 m where land is restricted.

NOTE: The absolute minimum width of median in urban area is 1.2 m and desirable minimum is 5.0 m.

The vital functions of the median are:

(*i*) To reduce the chances of head on collision between the vehicles moving in opposite directions.
(*ii*) To reduce the glaring effect of the head light of the vehicle moving in opposite direction in another lane.
(*iii*) At intersections, these provide a refuge for the cross-traffic.

Do you know? On long bridges and viaducts the width of median may be reduced to 1.5 m, but in any case this should not be less than 1.2 m.

3.3.5 Kerb

A vertical or sloping member along the edge of a pavement provided for supporting raised footpaths or central median is called Kerb. It is desirable to provide kerbs on urban roads.

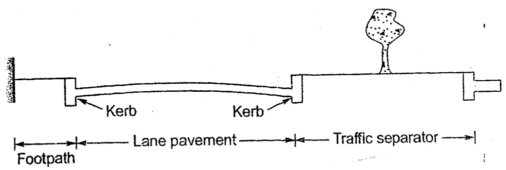


***Figure-3.6 :** Kerb and Traffic separator*

On the basis of their function kerbs are mainly divided into three groups:

(i) Low or Mountable type Kerb: Height of this type of kerb is about 10 cm with a slope or batter to help vehicles climb the kerb easily.

(ii) Semi - Barrier type Kerb: Height of this type of kerb is about 15 cm with a batter of 1 : 1 on the top 7.5 cm. It is provided where pedestrian traffic is high.

(iii) Barrier type Kerb: Height of kerb stone is about 20 cm with a batter of 1 : 4.

Functions of kerb are:

(*i*) To facilitate and control drainage.
(*ii*) To strengthen and protect the pavement edge.
(*iii*) To delineate the pavement edge.
(*iv*) To present a more finished appearance.
(*v*) To assist in the orderly development of the roadside.

3.3.6 Width of Pavement or Carriageway

The pavement width depends upon the number of lanes and width of a single lane required. The lane width is determined on the basis of the width of vehicle and minimum side clearance.

Maximum permissible width of vehicle as per IRC specifications is 2.44 m.

Table-3.11 : Width of Carriageway

Class of Road	Width of Carriageway
Single lane	3.75 m
Two lanes, without raised kerbs	7.0 m
Two lanes, with raised kerbs	7.5 m
Intermediate carriageway (except on important roads)	5.5 m
Multi-lane pavements	3.5 m per lane

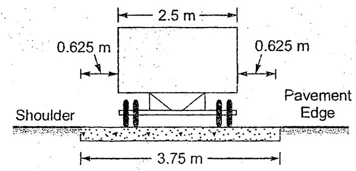


***Figure-3.7 :** Single lane Road with paved shoulders*

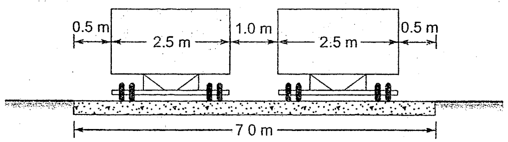


***Figure-3.8 :** Two lane road with single carriageway*

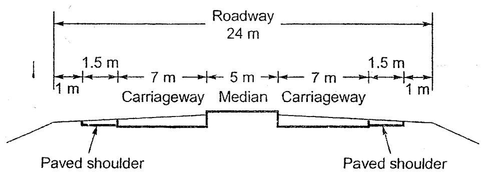


***Figure-3.9 :** Typical dual carriageway (2 lane each)*

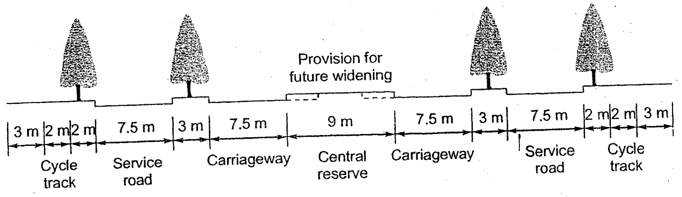


Figure-3.10 : *Urban arterial street*

NOTE: Intermediate carriageway is more than one lane but less than two lane. This is only provided to allow manouvering.

3.3.7 Camber or Cross Slope

Transverse slope provided to the road to drainoff rain water from road surface is known as camber. It is measured in 1 in 'n' or n% (e.g. 1 in 50 or 2%). Main advantage of providing camber are quick drying of pavement which in turn increases safety and subgrade protection by drainage.

Requirement of camber of a pavement depends upon:

(*i*) Type of pavement surface

(*ii*) The amount of rainfall

Table-3.12 : Camber for different road surfaces

Surface type	Camber
High type bituminous surfacing or cement concrete	1.7 to 2.0% (1 in 60 to 1 in 50)
Thin bituminous surfacing	2.0 to 2.5% (1 in 50 to 1 in 40)
Water bound macadam	2.5 to 3.0% (1 in 40 to 1 in 33)
Earth	3.0 to 4.0% (1 in 33 to 1 in 25)

Camber may be provided in the shape of parabolic, elliptic or straight line shape as shown in Figure 3.11. For cement concrete pavement straight line camber is provided while parabolic and elliptic shape are preferred by fast moving vehicles because they require frequent crossing of crown line during over taking operation.

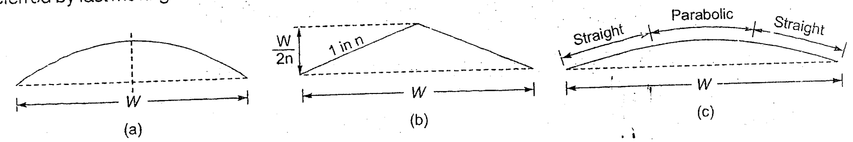


Figure-3.11 : *Shapes of cross slope*

The camber of shoulder should be atleast 0.5% steeper than the cross slope of adjoining pavement, subjected to a minimum of 3.0% and a maximum value of 5.0%.

Disadvantages of excessive camber are:

(*i*) Transverse tilt of vehicle and discomfort

(*ii*) Formation of cross ruts or depression

(*iii*) Center line tendency of vehicles in the movement

NOTE: Gradient provide for pavement is considered as twice of camber.

Example-3.1 In a district road 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 crown with respect to the edges in these two cases.

Solution:

(*i*) For WBM road

As the rainfall is heavy, provide a camber of 1 in 33, $\tan\theta = \frac{1}{33} = \frac{h}{\left(\frac{3.8}{2}\right)}$

$\therefore$ Rise of camber with respect to edges, $h = \left(\frac{3.8}{2}\right) \times \frac{1}{33} = 0.058$ m

(*ii*) For bituminous concrete road

Provide a cross-slope of 1 in 50, $\tan\theta = \frac{h}{\left(\frac{7}{2}\right)} = \left(\frac{1}{50}\right)$

Rise of crown with respect to the edges, $h = 3.5 \times \left(\frac{1}{50}\right) = 0.07$ m

3.3.8 Pavement Characteristics

The pavement surface depends on the type of availability of materials, traffic volume, subgrade and climatic conditions. The important pavement characteristics are friction, unevenness, light reflecting characteristics and drainage of surface water.

Friction

Friction between vehicle tyre and pavement surface is an important factor to determine the operating speed and stopping sight distance. Lack of adequate friction causes skidding or slipping.

Skidding takes place when the longitudinal movement of wheel is more than the rotational movement. To avoid skidding, frictional resistance must be greater than the breaking force.

Slipping occurs when wheel revolves more than the corresponding longitudinal movement along the roads. It generally occurs on pavement surface which is either slippery and wet or when the road surface is loose with mud.

Remember: As per IRC, coefficient of longitudinal friction lies between 0.35 - 0.40, used in sight distance calculation and coefficient of lateral friction is 0.15, used in horizontal curve design.

The various factors that affect friction are:

(*i*) Type of the pavement (like bituminous, concrete or gravel)

(*ii*) Condition of the pavement (dry or wet)

(*iii*) Condition of tyre (old or new)

(*iv*) Speed and load of the vehicle

NOTE: Smooth and worn out tyre offer higher friction factors on dry pavements than new tyres with treads due to large areas of contact but on wet pavements new tyres have higher friction factors.

Unevenness

Uneven surface increases riding discomfort, fuel consumption, wear and tear of tyres and other parts and accidents. The pavement surface conditions are commonly measured by using an equipment called "BUMP INTEGRATOR" in terms of unevenness index as given in **Table 3.13**, which is the cumulative measure of vertical undulations of the pavement surface recorded per unit horizontal length of the road.

$$\text{Bump indicator} = 630\,(\text{IRI})^{1.12}$$

where, IRI = International Roughness Index

Table-3.13 : Riding quality of pavement

Unevenness index (cm/km)	Riding quality
In old pavements	
Below 95	Excellent
95 to 119	Good
120 to 144	Fair
145 to 240	Poor (possible resurfacing)
Above 240	Very poor (resurfacing required)
In new pavements	
Below 120	Good (acceptable)
120 to 145	Fair (acceptable)
Above 145	Poor (not acceptable)

NOTE: For a good pavement, unevenness index should not be greater than 150 cm/km.

Light Reflecting Characteristic

Visibility at night is very much dependent upon the light reflecting characteristics of the pavement surface. Light coloured pavement surface gives goods visibility at night but produces glare and eye strain during the day light while on the other hand black top pavement provides poor visibility in night. Therefore we provide the black top pavement with the white borders. White borders guide the movement of vehicles in night time.

Drainage

The pavement must be completely impermeable so that seepage of water into the pavement layers does not takes place and the geometry of pavement surface should be such that which helps in draining out the water from the surface quickly.

3.4 Sight Distance

Sight distance is the distance along a road at which a driver from a specified height above the carriageway has a visibility of objects and can safely stop his vehicle or overtake another vehicle. Sight distance requirements is needed in the design of vertical curves and it also governs the set back distances of buildings or any other obstructions adjacent to the carriageway on a horizontal curve.

Sight distances considered by IRC in highway design are:

(*i*) Stopping Sight Distance (SSD)/Non-passing sight distance
(*ii*) Overtaking Sight Distance (OSD)/Passing sight distance
(*iii*) Safe Sight Distance required for entering in an intersection
(*iv*) Intermediate Sight Distance (ISD)
(*v*) Head Light Sight Distance (HSD)

Sight distance on a highway depends upon the following factors:

(i) Speed of Vehicle: Speed of a vehicle is directly related with the sight distance. More will be the speed of vehicle, more will be the sight distance required.

NOTE: If speed of vehicle is not mentioned in the problem then considered the design speed of highway as the speed of vehicle.

(ii) Driver's Reaction Time: As we discussed earlier, reaction time is the time taken by the driver from the instant of seeing the object to the instant when brakes are applied. Total reaction time is measured on the basis of PIEV theory which varies from 0.5 seconds for simple situations to 3 - 4 seconds for complex situations.

Remember: IRC recommends, reaction time of 2.5 seconds for stopping sight distance and 2.0 seconds for overtaking sight distance calculations.

(iii) Brake's Efficiency: Efficiency of brakes depends upon the age and characteristics of vehicle. If we say the brakes are 100% efficient, it means vehicle will stop at the moment of application of brakes. But practically 100% efficiency of brakes is not achieved, otherwise skidding will occur which is uncontrollable and dangerous for the vehicle and road users.

NOTE: For the design purpose of highway, we considered only 50% brake efficiency of vehicle.

(iv) Gradient of Pavement: When we are going down on a gradient, gravitational force also comes into action which causes the vehicle to take more time to stop the vehicle means more sight distance is required. While in climbing up a gradient less sight distance is required because time taken to stop the vehicle will be less.

(v) Frictional Resistance: Less distance is required by vehicle to stop when frictional resistance is more, but more friction will cause more wear and tear in the tyre which is not beneficial for the vehicle. That's why IRC recommend the value of longitudinal friction in between 0.35 to 0.40.

Table-3.14 : Coefficient of longitudinal friction

Speed (kmph)	20 - 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

3.4.1 Stopping Sight Distance

Stopping sight distance is the minimum distance over which the driver travelling at design speed can apply brakes and bring the vehicle to stop position safely without collision with any other obstruction. It is also known as minimum sight distance or non passing sight distance. Stopping sight distance should be provided throughout the length of all roads.

Stopping sight distance composed of two components:

(i) Lag Distance (d): It is the distance travelled by the vehicle in total reaction time, if v is the design speed in m/s and t is the total reaction time in sec, then

$$\text{lag distance } (d) = vt \text{ metres}$$

if V is design speed in kmph and t in seconds, then

$$d = 0.278\,Vt \text{ metres}$$

(ii) Braking Distance (l): It is the distance travelled by vehicle after the application of brakes. It is obtained by equating the work done in stopping the vehicle and kinetic energy stored in the vehicle. Let us consider, F is the maximum frictional force developed and l is the braking distance, then the workdone against friction in stopping the vehicle will be $F\,l$.

where, frictional force, $F = fW$

where, 'W' is the total weight of vehicle and 'f' is the coefficient of friction

So, workdone in stopping the vehicle = fWl ...(i)

while the kinetic energy stored in vehicle $= \frac{1}{2}mv^2 = \frac{1}{2}\frac{Wv^2}{g}$ $(\because W = mg)$ ...(ii)

Now by using the law of conservation of energy,

Workdone in stopping the vehicle = Kinetic energy stored in vehicle

$$\Rightarrow \quad fWl = \frac{Wv^2}{2g}$$

$$\therefore \quad l = \frac{v^2}{2gf}$$

NOTE: If braking efficiency is η, the braking distance $l = vt + \frac{v^2}{2gf\eta}$

Therefore,

Stopping Sight Distance(SSD) = Lag distance + Braking distance

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

where, 'v' is design speed in m/s, 'g' is the acceleration due to gravity in m/s^2, 't' is the total reaction time in sec and 'f' is coefficient of friction.

The values of stopping sight distance recommended by IRC on the basis of design speed are given in Table 3.15.

Table-3.15 : Stopping sight distance as per Indian practice

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

Example-3.2 Calculate the safe stopping sight distance for design speed of 80 kmph for:

(*i*) Two-way traffic on a two lane road

(*ii*) Two-way traffic on a single lane road

(Consider braking efficiency as 50%)

Solution:

Consider total reaction time, $t = 2.5$ seconds and longitudinal coefficient of friction, $f = 0.35$

SSD = Lag distance + Braking distance

$$SSD = vt + \frac{v^2}{2gf\eta} \quad ...(i)$$

Given, $V = 80$ kmph or $v = \frac{80}{3.6} = 22.22$ m/s

From equation (*i*), $SSD = 22.22 \times 2.5 + \frac{(22.22)^2}{2 \times 9.81 \times 0.35 \times 0.5} = 55.55 + 143.83 = 199.38$ m

(*i*) Stopping sight distance when there are two lanes = 199.38 m

(*ii*) Stopping sight distance for two way traffic with single lane = 2 × SSD = 2 × 199.38 = 398.76 m

Remember: If width is restricted on road or on single lane road when two way movement of traffic is permitted, then the minimum stopping sight distance will be twice of stopping sight distance.

Effect of Ascending Gradient on Braking Distance

Equating the kinetic energy of vehicle and workdone to stop it

$$\Rightarrow \quad (fW + W\sin\theta)l = \frac{Wv^2}{2g} \quad ...(i)$$

Suppose ascending gradient of say n,

for very small θ, $\sin\theta \simeq \tan\theta \simeq \frac{n}{100}$

Equating this value of $\sin\theta$ in equation (*i*),

$$\Rightarrow \quad \left(fW + \frac{Wn}{100}\right)l = \frac{Wv^2}{2g}$$

$$\Rightarrow \quad l = \frac{v^2}{2g\left(f + \frac{n}{100}\right)} = \frac{v^2}{2g(f + 0.01n)}$$

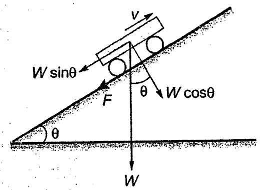


***Figure-3.12 :** Vehicle on ascending gradient*

Hence, in case of ascending gradient the braking distance required is less.

Effect of Descending Gradient on Braking Distance

Similarly as in the previous case, braking distance is given as

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

or

$$l = \frac{v^2}{2g(f - 0.01n)}$$

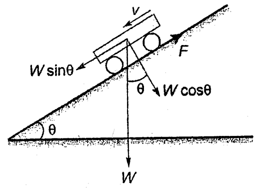


***Figure-3.13 :** Vehicle on descending gradient*

Hence on the descending gradient, the braking required is more.

Now in the most generalised form, the expression for the calculation of stopping sight distance is given as

$$SSD = vt + \frac{v^2}{2g(f\eta \pm 0.01n)}$$

where, v = Design speed in m/s, t = Reaction time in sec

g = Acceleration due to gravity in m/s^2

f = Coefficient of friction

n = Gradient, η = Braking efficiency

Example-3.3 Calculate the stopping sight distance on a highway at a descending gradient of 3% for a design speed of 100 kmph. Assume other data as per IRC recommendations.

(a) 126.18 m (b) 192.40 m
(c) 252.36 m (d) 384.80 m

Ans. (b)

Total reaction time, *t*, taken as 2.5 seconds and coefficient of friction, *f* as 0.35

V = 100 kmph, $n = -3\% = -0.03$ and $g = 9.81$ m/s²

$$v = \frac{100}{3.6} = 27.78 \text{ m/s}$$

$$\Rightarrow \quad SSD = vt + \frac{v^2}{2g(f - 0.01n)}$$

$$\Rightarrow \quad SSD = 27.78 \times 2.5 + \frac{(27.78)^2}{2 \times 9.81(0.35 - 0.01 \times 3)}$$

$$\therefore \quad SSD = 69.45 + 122.92 = 192.37 \text{ m}$$

Example-3.4 Calculate the safe stopping distance while travelling at a speed of 80 kmph on an upward gradient of 2 percent. Make suitable assumptions.

Solution:

(*i*) Perception and brake reaction time : 2.5 sec (as per IRC)

(*ii*) Coefficient of friction : Varying from 0.40 at 20 kmph to 0.35 at 100 kmph (Table 3.14)

$$\text{Coefficient of friction,} \quad f = 0.35 + \frac{0.05 \times (100 - 80)}{(100 - 20)} = 0.35 + 0.01 = 0.36$$

$$\text{Stopping distance,} \quad d = 0.278\,Vt + \frac{V^2}{254(f + 0.01n)}$$

$$= 0.278 \times 80 \times 2.5 + \frac{80 \times 80}{254(0.36 + 0.01 \times 2)}$$

$$= 55.6 + \frac{6400}{254 \times 0.38} = 55.6 + 66.3 = 121.9 \text{ m}$$

3.4.2 Overtaking Sight Distance (OSD)

Overtaking is a necessary operation because all the vehicles do not travel with a uniform speed. Overtaking is only possible when driver has sufficient sight distance to complete the whole operation.

It is the minimum distance visible to the driver of a vehicle who is intending to overtake the slow moving vehicle ahead safely against the traffic of opposite direction. It is measured along the centre line of road over which driver with his eye level 1.2 m above the pavement surface can see the top of an object of height 1.2 m above the road surface as shown in Figure 3.14.

NOTE: Generally in overtaking operations, a vehicle overtakes an another vehicle, not any other object, so height of object is fixed to 1.2 m above road surface.

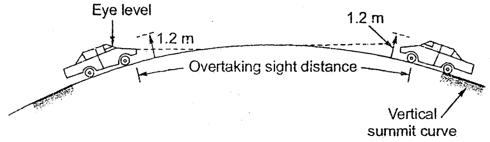


***Figure-3.14 :** Measurement of overtaking sight distance*

Factors affecting the overtaking sight distance:

(*i*) Speed of overtaking, overtaken and vehicle coming from opposite direction

(*ii*) The spacing between overtaking and overtaken vehicle

(*iii*) Skill and reaction time of driver

(*iv*) Rate of acceleration of overtaking vehicle

(*v*) Gradient of road

Analysis of Overtaking Sight Distance

Let us suppose, Vehicle A = Travelling at the design speed of highway
Vehicle B = Slow moving vehicle which is going to be overtaken by vehicle A
Vehicle C = Travelling in the opposite direction of vehicle A and B

The whole process of overtaking is split into three parts d_1, d_2 and d_3 as shown in Figure 3.15.

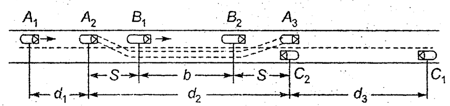


***Figure-3.15 :** Overtaking manoeuvre*

Where, d_1 = Distance travelled by overtaking vehicle A during the reaction time t sec.
d_2 = Distance travelled by the vehicle A during the actual overtaking operation in time T seconds.
d_3 = Distance travelled by the vehicle C comes from the opposite direction.
V_b = Speed of the slow moving vehicle (in kmph)

NOTE

- IRC recommends the total reaction time (t) of 2 seconds for overtaking sight distance.
- If v_b value is not given, then assume
  $v_b = v - 4.5$ m/s
  $V_b = V - 16$ kmph
  where v or V is the design speed in m/s or kmph.

During calculation of d_1, d_2 and d_3 we made following assumptions:

(*i*) It is assumed that the vehicle A is forced to reduce its speed to the speed of slow moving vehicle (v_b).

(*ii*) Spacing between vehicle A and vehicle B is s, till there is an opportunity for safe overtaking operation and distance travelled by vehicle A during this reaction time is d_1.

Now 'd_1' is the distance travelled by the vehicle A in the reaction time, which is given as

$$d_1 = v_b t \text{ metres}$$

where, v_b = Speed of slow moving vehicle in m/s

Spacing between the vehicle A and B just before the starting of overtaking operation (i.e. at position A_2 and B_1) when both the vehicles moving with the speed of v_b m/s is given by an empirical formula

$$S = 0.7\ v_b + \text{length of vehicle}$$

Generally, length of vehicle considered is 6 m

$$S = 0.7\ v_b + 6 \text{ metres} \quad ...[v_b \text{ in m/s}]$$

or

$$S = 0.2\ V_b + 6 \text{ metres} \quad ...[V_b \text{ in kmph}]$$

Let, 'b' is the distance travelled by vehicle B in the complete overtaking operation time, 'T' sec.

$$b = v_b T \text{ metres}$$

$$d_2 = b + 2S \text{ metres} \quad ...(i)$$

and

$$d_2 = v_b T + \frac{1}{2} a T^2 \text{ metres} \quad ...(ii)$$

equating equation (i) and (ii), we get

$$b + 2S = v_b T + \frac{1}{2} a T^2 \Rightarrow v_b T + 2S = v_b T + \frac{1}{2} a T^2$$

$$\Rightarrow \quad 2S = \frac{1}{2} a T^2$$

$$\therefore \quad T = \sqrt{\frac{4S}{a}} \quad \text{(where, } a \text{ is in m/s}^2\text{)}$$

or

$$T = \sqrt{\frac{14.4S}{A}} \quad \text{(where, } A \text{ is in kmph/sec)}$$

Table-3.16 : Maximum overtaking acceleration at different speeds

Speed		Maximum overtaking acceleration	
V (kmph)	**V (m/s)**	**A (kmph/sec)**	**a (m/s²)**
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

Now, $d_2 = v_b T + 2S$ metres

The distance travelled by vehicle C moving at design speed v m/s during the overtaking operation of A

$$d_3 = vT \text{ metres}$$

Therefore,

$$\text{OSD} = d_1 + d_2 + d_3 \text{ metres}$$

$$\text{OSD} = v_b t + v_b T + 2S + vT \text{ metres} \quad (v_b \text{ and } v \text{ in m/s, } t \text{ and } T \text{ in sec})$$

NOTE: On divided highways with one way traffic the overtaking distance need to be only ($d_1 + d_2$), because no vehicle is expected from the opposite direction.

The values of overtaking sight distance recommended by IRC are given in **Table 3.17**.

Table-3.17 : Overtaking sight distance

Speed(kmph)	40	50	60	65	80	100
Safe Overtaking Sight Distance (metres)	165	235	300	340	470	640

Effect of Gradient on Overtaking Sight Distance

On an ascending gradient, the overtaking sight distance required is more due to reduced acceleration of overtaking vehicle and vehicle coming from opposite direction is likely to speed up. Effect of gradient is compensated by reducing the speed of overtaken vehicle.

Overtaking Zone

It is desirable to construct highway such that the length of road visible ahead is sufficient for safe overtaking operation. But practically seldom it is not possible to provide overtaking sight distance throughout the length. In that case we provide particular zones marked with wide road for overtaking operation in the highway. These zones which are meant for overtaking are called overtaking zones.

The desirable length of overtaking zones is kept five times the overtaking sight distances.

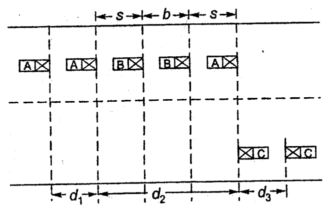
NOTE: Minimum length of overtaking zone is taken as 3 times of overtaking sight distance.

Example - 3.5 On a two way traffic road, the speed of overtaking vehicles are 100 kmph and 50 kmph. If the average acceleration is 0.92 m/s². Determine the overtaking sight distance. Draw a neat-sketch of the overtaking zone and show the positions of the sign posts.

Solution:

Overtaking sight distance for a two way traffic

$$= d_1 + d_2 + d_3$$



$$V_b = 50 \text{ kmph}$$

or

$$\frac{50}{3.6} = 13.89 \text{ m/sec}$$

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

or

$$\frac{100}{3.6} = 27.78 \text{ m/sec}$$

$$a = 0.92 \text{ m/s}$$

$$t = 2.0 \text{ sec} \quad \text{(Assume)}$$

$$\therefore \quad d_1 = v_b t = 13.89 \times 2 = 27.78 \text{ m}$$

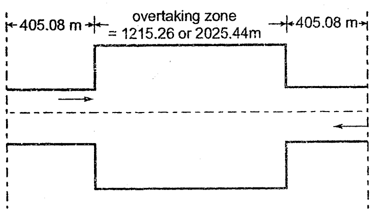
Now,

$$d_2 = b + 2S = v_b T + 2S \quad ...(i)$$

$$S = 0.7\ v_b + 6 = 0.7 \times 13.89 + 6$$

$$S = 15.723 \text{ m}$$

$$T = \sqrt{\left(\frac{4S}{a}\right)} = \sqrt{\frac{4 \times 15.723}{0.92}} = 8.3 \text{ s}$$



using equation (*i*),

$$d_2 = 13.89 \times 8.3 + 2 \times 15.723$$

and $$d_2 = 146.733 \text{ m}$$

$$d_3 = v_c t = 27.78 \times 8.3$$

$$d_3 = 230.57 \text{ m}$$

$$\text{Total OSD} = d_1 + d_2 + d_3$$

$$= 27.78 + 146.733 + 230.57 = 405.08 \text{ m}$$

Minimum length of overtaking zone = 3 × OSD = 3 × 405.08 = 1215.24 m

Desirable length of overtaking zone = 5 × OSD = 5 × 405.08

= 2025.4 m

3.4.3 Sight Distance at Intersection

At intersections where two or more road meets, clear view across the corners should be provided from a sufficient distance to avoid collision of vehicles. The sight distance should be provided in such a way so that driver from both sides can see each other.

The area of unobstructed sight formed by the lines of vision is called the sight triangle as shown in Figure 3.16.

The design of sight distance at intersection will be based on three possible conditions:

(*i*) Enabling approaching vehicle to stop

(*ii*) Enabling the approaching vehicle to change speed

(*iii*) Enabling stopped vehicle to cross a main road

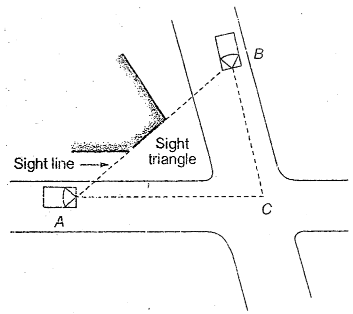


***Figure-3.16 :** Sight distance at intersection*

NOTE

IRC recommends that at uncontrolled intersections, sufficient visibility should be provided such that the sight distance of each road is atleast equal to the stopping sight distance corresponding to the design speed of the road.

3.4.4 Intermediate Sight Distance

On a horizontal curve, the requirement of overtaking sight distance can not always be satisfied. In such cases overtaking is prohibited by using regulatory signs. To provide an opportunity for overtaking operation on horizontal curves or in restricted areas, we provide intermediate sight distance, i.e. equals to twice of stopping sight distance.

Table-3.18 : Intermediate sight distance

Speed (kmph)	20	25	30	35	40	50	60	65	80	100
Intermediate sight distance (m)	40	50	60	80	90	120	160	180	240	360

NOTE: The measurement of ISD is made assuming both the height of the eye level of the driver and the object to be 1.2 m above the road surface.

3.5 Curve

Curves are provided in highways in order that the change of direction at the intersection of straight alignments either in horizontal or vertical plane, shall be gradual.

The necessity of curves arises due to the following reasons:

(*i*) Topography of the country

(*ii*) To provide access to a particular locality

(*iii*) Restrictions imposed by property

(*iv*) Preservation of existing amenities

(*v*) Avoidance of existing religious, monumental and other costly structures

(*vi*) Making use of existing right of way

3.5.1 Advantages of Curves

The advantages of providing curves are:

(*i*) They provide comfort to the passengers. If there is an abrupt change in the direction nor grade of a highway it will upset the passengers.

(*ii*) They help to avoid mental strain induced by the monotony of continuous journey along straight path.

(*iii*) In the case of sharp turns, brakes have to be applied more frequently which reduces the life of tyres. Thus life of the vehicles in increased by providing curves.

(*iv*) The drivers become alert due to the change in the direction of road.

(*v*) They help to keep the speed of the vehicle within limits. On a straight road, a driver is tempted to go at a much faster speed.

3.5.2 Factor Affecting the Design of Curves

The various factors which affect the design of curves are:

(*i*) Design speed of the vehicle

(*ii*) Allowable friction

(*iii*) Maximum permissible superelevation

(*iv*) Permissible centrifugal ratio

3.5.3 Type of Curves

Curves have been divided into two classes:

(i) **Horizontal Curve:** A curve in plan to provide change in direction of the central line of a road.

(ii) **Vertical Curve:** A curve in the longitudinal section of a roadway to provide for easy change of gradient.

The different types of curves used in highways are:

(i) **Simple Circular Curve:** A simple curve consists of a single are connecting two straights.

(ii) **Compound Curve:** A compound curve consists of a series of two or more simple curves that turn in the same direction and join at common tangent points.

(iii) **Reverse Curve:** A reverse curve consists of two simple curves of opposite direction that join at the common tangent point called the point of reverse curve.

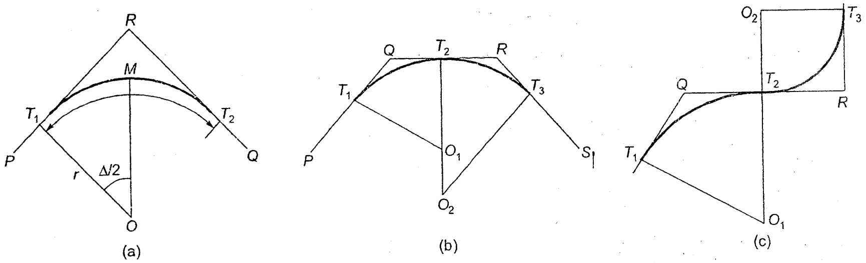


Figure-3.17 : (a) Simple circular curve (b) Compound curve and (c) Reverse curve

3.6 Design of Horizontal Alignment

Centrifugal force acted on the vehicle depends upon speed and radius of horizontal curves and it is counteracted upto a certain extent by the frictional force between the tyre and pavement surface.

When a vehicle moves on a horizontal curve then it is subjected to an outward force, commonly known as centrifugal force acting in outward direction which causes discomfort to the passenger and increases the possibility of accidents. So, design of horizontal alignment is an important feature which influence the efficiency and safety of highway. The design speed is the single most important factor which governs the design of horizontal alignment.

In order to resist the centrifugal force, it is usual practice to super-elevate the roadway cross-section. The horizontal alignment design elements include radius of circular curve, superelevation, extra widening, transition curves and set back distance.

3.6.1 Stability Analysis on Horizontal Curve without Superelevation

Figure 3.18 (b) shows the forces acting on the vehicle at a non super-elevated horizontal curve section.

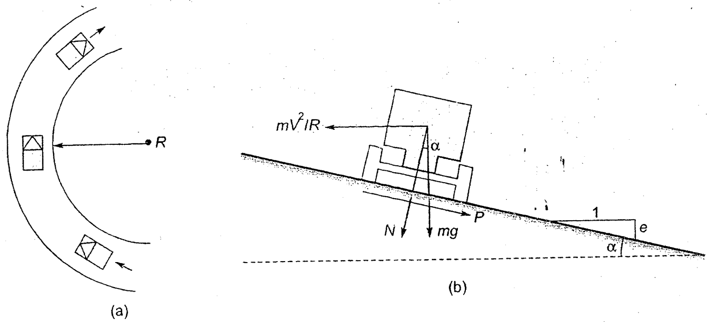


Figure-3.18 : Forces acting on vehicle on a curve

Suppose a vehicle of mass 'm' moves on a curve of Radius 'R' with a speed of 'v', then the centrifugal force 'P' acting on the vehicle is

$$P = \frac{mv^2}{R} = \frac{Wv^2}{gR} \qquad \left(\because\ m = \frac{W}{g}\right)$$

The centrifugal force acting on a vehicle running on a horizontal curves has two effects:

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

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

The ratio of centrifugal force to the weight of vehicle $\left(\frac{P}{W}\right)$ is known as the centrifugal ratio or impact factor i.e. $\frac{P}{W} = \frac{v^2}{gR}$

Stability Condition Against Overturning

The centrifugal force 'P' tends to overturn the vehicle about the outer wheels B of the vehicles moving on a horizontal curve as shown in **Figure 3.19**.

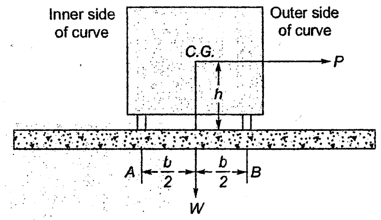


Figure-3.19 : Overturning due to centrifugal force

Let, W = Weight of the vehicle

v = Speed of the vehicle in m/s

= 0.28 V, where V is in kmph

R = Radius of the horizontal curve in m

P = Centrifugal force acting on the vehicle

R_A and R_B = Normal Reaction at A and B to the surface

h = Height of centre of gravity of vehicle above road level

b = Centre to centre distance between the wheels

Now for the stability against overturning, the limiting condition is,

Overturning Moment due to centrifugal force ≤ Restoring moment due to weight of vehicle

$$P \times h \le W \times \frac{b}{2}$$

$$\frac{P}{W} \le \frac{b}{2h}$$

Hence, to avoid the overturning the centrifugal ratio should be less than or equal to $\frac{b}{2h}$.

NOTE For the stability against overturning on horizontal curve we consider that the vehicle is at the verge of overturning i.e. the normal reaction at the wheels A is zero, therefore its effect is neglected in accounting of overturning moment.

Stability Condition Against Transverse Skidding

Centrifugal force developed also has the tendency to push the vehicle outwards in transverse direction. The equilibrium condition to resist the transverse skidding of the vehicle is given by

$$P \le F_A + F_B$$

$$\Rightarrow \quad P \le f(R_A + R_B) \le fW$$

where, f = Coefficient of friction between tyre and pavement surface

$$\therefore \quad \frac{P}{W} \le f$$

Remember: Thus to avoid overturning and transverse skidding on a horizontal curve, the centrifugal ratio should always be less than $b/2h$ and f.

3.6.2 Impact Factor

This is defined as the ratio of the maximum centrifugal force to the weight of the vehicle and is expressed as a percentage. The centrifugal force P, is given by

$$P = \frac{W}{g} \times \frac{v^2}{r}$$

where, W = Weight of the vehicle in kg

v = Speed in m/s

r = Radius of curvature in m at that particular instant

Centrifugal force is maximum when r is minimum, which occurs at the end of the transition curve i.e. when $r = R$.

$$\therefore \quad P_{max} = \frac{W}{g} \times \frac{v^2}{R}$$

If I is the impact factor, then

$$I(\%) = \frac{P_{max}}{W} \times 100 = \frac{v^2}{gR} \times 100$$

As $R = \dfrac{L}{2N}$ and $v = 0.28\ V$, where V is in kmph

$$\therefore \quad I(\%) = \frac{(0.28V)^2}{9.81 \times \dfrac{L}{2N}} \times 100 = 1.59\frac{NV^2}{L}$$

3.6.3 Superelevation

In passing from a straight to a curved path, a vehicle is under the influence of two forces, namely

(*i*) the weight of the vehicle

(*ii*) the centrifugal force

Both the forces acting through its centre of gravity. The centrifugal force always acts in the horizontal direction.

Raising of outer edge of the pavement with respect to inner edge throughout the length of curve to counteract the centrifugal force and reduce the tendency of vehicle to overturn is called superelevation. Superelevation on curves is intended to counteract a part of the centrifugal force, the remaining part being resisted by the lateral friction while it can be provided to fully counteract the centrifugal force also.

The superelevation 'e' is expressed as the ratio of the height of outer edge with respect to the horizontal width.

From Figure 3.20 it may be seen that the superelevation,

$$e = \frac{PQ}{QR} = \tan\theta$$

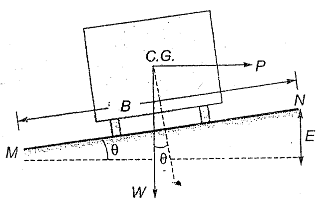


Figure-3.20 : Superelevated pavement section

In practise the inclination 'θ' to the horizontal is very small. Therefore,

$$e = \tan\theta \simeq \sin\theta = \frac{E}{B}$$

It is measured as the ratio of the relative elevation of the outer edge i.e. E to width of pavement B.

Relation Between Superelevation, Coefficient of Friction and Centrifugal Ratio

To counteract the effect of the centrifugal force, let the surface of the road be superelevated by an angle 'θ' to the horizontal. Figure 3.21 shows the vehicle is moving perpendicular to the plane of the paper.

Where, W = Weight of vehicle

F = Friction force between the wheels and pavements

θ = Transverse slope due to superelevation

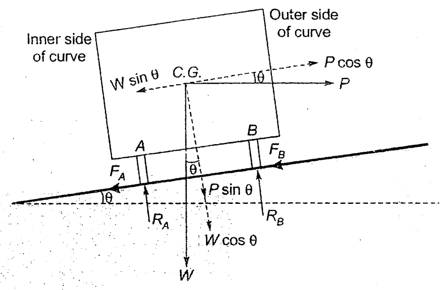


Figure-3.21: Analysis of Superelevation

For the equilibrium consideration, the summation of forces parallel to the pavement surface is taken to be zero. Therefore, we get

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

$$\Rightarrow \quad P\cos\theta = W\sin\theta + f(R_A + R_B) \qquad (\because F_A = fR_A \text{ and } F_B = fR_B)$$

$$\Rightarrow \quad P\cos\theta = W\sin\theta + f(W\cos\theta + P\sin\theta)$$

Now divide the whole equation by Wcosθ, we get

$$\Rightarrow \quad \frac{P}{W} = \tan\theta + f + f\frac{P}{W}\tan\theta$$

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

Now, for small 'θ', $\tan\theta = e$

$$\Rightarrow \quad \frac{P}{W} = \frac{e+f}{1-ef}$$

Now, according to permissible value of 'e' and 'f', $1 - ef \simeq 1$

Hence the above equation is modified as

$$\therefore \quad \frac{P}{W} = e + f \qquad ...(i)$$

and we also know that the centrifugal ratio can also be given as

$$\frac{P}{W} = \frac{v^2}{gR} \qquad ...(ii)$$

Equating equation (i) and (ii), we get

$$e + f = \frac{v^2}{gR} \qquad \text{(Here, '}v\text{' is in m/s and '}R\text{' in m)}$$

if v is expressed as V in kmph then the equation is changed into, $e + f = \dfrac{V^2}{127R}$

This is a fundamental equation for the horizontal curve design.

Guidelines Recommended by IRC for Superelevation

(i) Superelevation in the road is design for a particular vehicle which has standard weight and dimensions.

(ii) The maximum value of superelevation is governed in India by the bullock cart traffic since these will be using the roads for a considerable time to come.

(iii) Maximum superelevation provided for a particular type of road is given in Table 3.19.

Table-3.19 : Maximum superelevation

Type of Road	Urban Road	Plain and Rolling terrain	Hilly Regions
Maximum Superelevation (e_{max})	4 % or 0.04	7% or 0.07	10% or 0.10

NOTE: The minimum superelevation provided on a road must not be less than the camber of roads for the adequate drainage of water from the surface.

Value of Coefficient of Lateral Friction

The value of the coefficient of lateral friction depends upon a number of factors:

(i) Speed of vehicle

(ii) Type and condition of roadway surfaces

(iii) Type and condition of the tyres

The value of 'f' varies from 0.20 for muddy roads on a rainy day to 0.15 for newly constructed asphalt roads. Since the condition of Indian roads is not good, the IRC has recommended the least value of 0.20. Taking a factor of safety of $\frac{4}{3}$, the value of safe coefficient of friction works out to be $0.2 \times \frac{3}{4} = 0.15$.

Equilibrium Superelevation

Equilibrium superelevation is that superelevation at which pressure at the inner and outer tyre will be equal. If the coefficient of friction is neglected i.e. $f = 0$, then

$$\tan\theta = e = \frac{V^2}{127R} \quad \text{and} \quad \theta = \tan^{-1}\left(\frac{V^2}{127R}\right)$$

This means that if a road surface is superelevated by $\tan^{-1}\left(\frac{V^2}{127R}\right)$ then the frictional force will not be called upon to act and thus the pressure on both the wheels will be equal.

$$\therefore \quad e_{\text{equilibrium}} = \frac{v^2}{gR} = \frac{V^2}{127R}$$

The distribution of pressure in the case of a superelevated and level road is shown in Figure 3.22 (b).

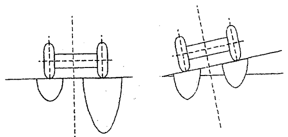


Figure-3.22 : *Distribution of pressure on wheels on (a) Level road (b) Superelevated*

If the superelevation provided is less than $e_{\text{equilibrium}}$, lateral friction will come into play and will go on increasing as the superelevation gets reduced.

Effect of Superelevation on Passengers

So far, the effect of superelevation on the vehicle has been considered, where the lateral thrust is restricted by the lateral friction between the tyres and the road surface. In case of passengers that lateral thrust may tilt the passengers in their seats.

The forces acting on the passenger while the vehicles is moving on the curve as shown in the Figure 3.23.

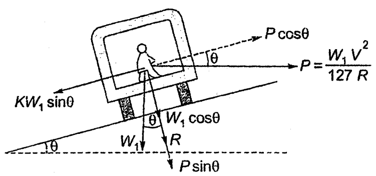


Figure-3.23 : *Forces acting on a passenger, when the vehicle moves on a curve*

Where, W_1 = Weight of passenger

K = Lateral ratio

Resolving the force along the inclined plane, the net force KW_1, acting on the passenger is given by

$$KW_1 = P\cos\theta - W_1\sin\theta = \frac{W_1V^2}{127R}\cos\theta - W_1\sin\theta$$

or,
$$K = \frac{V^2}{127R}\cos\theta - \sin\theta$$

thus if
$$e = \tan\theta = \frac{V^2}{127R}$$

then,
$$K = \tan\theta\cos\theta - \sin\theta = \sin\theta - \sin\theta = 0$$

$\therefore$ When $e = \frac{V^2}{127R}$ the passenger will not feel that the vehicle is taking a turn.

Do You Know

If superelevation = $\frac{V^2}{127R}$ is provided on curves, the driver is not required to apply any force on the wheel. The speed at which this occurs is known as "Hand off speed". With the increase in the value of KW_1, the force required to be applied on the wheel also increases.

Hand off speed, $V = \sqrt{127R\sin\theta}$

Maximum Value of Centrifugal Ratio

Since a limit has been imposed on the maximum value of superelevation and lateral frictional resistance, the centrifugal force cannot be allowed to develop beyond a certain value. Thus the maximum centrifugal ratio, that can be permitted is determined from the limiting values of e and f i.e. $e_{max} = 0.07$ and $f_{max} = 0.15$

Maximum permissible centrifugal ratio (C) = $e_{max} + f_{max} = 0.07 + 0.15 = 0.22$

But also,
$$C = \frac{V^2}{127R}$$

$$\therefore \quad \frac{V^2}{127R} = 0.22 \quad \text{or} \quad R = \frac{V^2}{28} \qquad \text{(Approximately)}$$

Thus for any design speed, the minimum permissible radius of curvature is also fixed. Table 3.20 shows the radii of horizontal curves for different camber rates beyond which superelevation will not be required.

Table-3.20 : Radii beyond which superelevation is not required

Design speed (kmph)	Radii (metres) 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
65	470	620	750	950	1100
80	700	950	1100	1400	1700
100	1100	1500	1800	2200	2600

Attainment of Superelevation

Roads on straight path are usually provided with a camber on both sides from the crown or the central line of a road. But on curves, this road has to be superelevated, i.e., outer edge has to be raised with respect to the inner edge. Thus it is required to change the cambered section into a superelevated one.

Superelevation in road is accomplished in two stages:

(i) Elimination of crown of the cambered section: It is done by two methods:

(a) Rotation of outer edge about the crown as shown in Figure 3.24, at a desired rate such that the outer edge of the cross slope is brought upto the level or horizontal at the start of a transition curve.

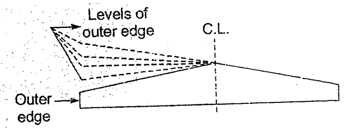


Figure-3.24 : Outer edge rotated about the crown

(b) In the second method of elimination of crown, the crown is progressively shifted outwards as shown in Figure 3.25, thus the width of inner edge increases progressively. This method is known as diagonal crown method. But this method is not usually adopted due to the increase in negative superelevation on to a portion of the outer half.

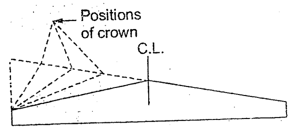


Figure-3.25 : Crown shifted outwards (diagonal crown method)

(ii) Rotation of Pavement to Attain full Superelevation:

There are two methods of attaining superelevation by rotating the pavement:

(a) **Rotating about centre line:** The pavement is rotated about the centre line such that inner edge is depressed and outer edge is raised both by an amount of $E/2$, where 'E' is total superelevation. In this method vertical profile of the centre remains unchanged and earthwork is balanced resulting in an advantage.

The main disadvantage of this method is the drainage problem because of the depressing inner edge.

Figure 3.27 illustrates this method diagrammatically. The small cross-sections at the bottom of each diagram indicate the pavement cross-slope condition at different points.

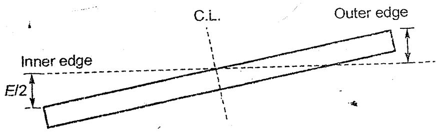


Figure-3.26 : Rotating about centre line

NOTE: In Hilly areas, rotation about centre line is generally preferred because earthwork is balance and in hilly region there is no drainage problem also.

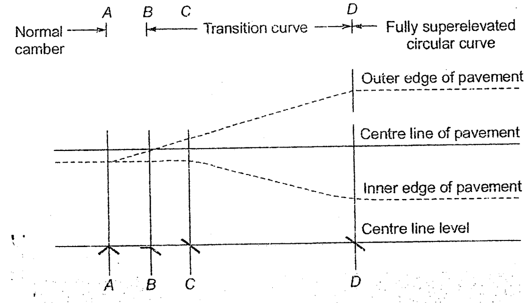


Figure-3.27 : Pavement revolved about centreline

NOTE: If subgrade is in embankment or in case of significant gradient for the longitudinal drainage, then there will be no drainage problem.

(b) **Rotating about inner edge:** In this method, pavement is rotated about inner edge until the attainment of full superelevation. This is more preferable in the very flat terrain to avoid the drainage problem. The entire width of pavement is raised by additional earth fill, which may be considered as a disadvantage of the method.

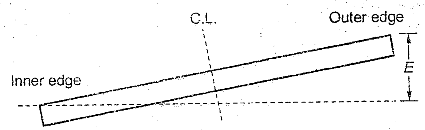


Figure-3.28 : Rotating about centre the inner edge

Figure 3.29 illustrates this method diagrammatically. The small cross-sections at the bottom of each diagram indicate the pavement cross-slope condition at different points.

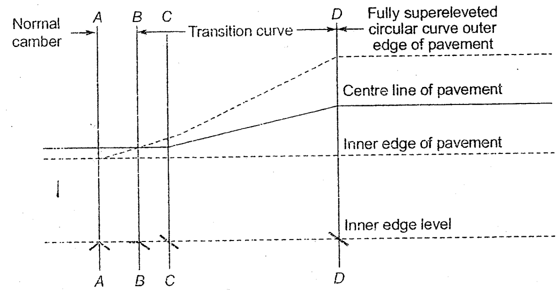


Figure-3.29 : Pavement revolved about inner edge

3.6.4 Design of Superelevation

Design of superelevation for mixed traffic condition is a complexed problem because different vehicles move with different speeds.

Maximum value of superelevation by neglecting the lateral friction is safer for fast moving vehicle but inconvenient for slower vehicle. While lower superelevation, depending more on later friction would be unsafe for faster vehicle.

So for practical condition IRC recommends that superelevation should be provided to fully counteract the centrifugal force due to 75% of design speed by neglecting the lateral friction.

Following design steps are involved into the design of superelevation:

Step-1: Calculate the superelevation corresponding to 75% of design speed and neglecting the role of lateral friction.

$$\Rightarrow \quad e + 0 = \frac{(0.75\,V)^2}{127R}$$

$$\therefore \quad e_{equilibrium} = \frac{V^2}{225R} \qquad \text{(Here, } V \text{ is in kmph)}$$

If $e_{calculated} < e_{maximum}$, then provide $e = e_{equilibrium}$

If $e_{calculated} > e_{maximum}$, then take $e = e_{maximum}$

Step-2: Provide $e = e_{maximum}$, and find the value of lateral friction 'f'

$$\Rightarrow \quad e + f = \frac{V^2}{127R}$$

$$\therefore \quad f = \frac{V^2}{127R} - e_{maximum}$$

If, $f < 0.15$, then provide $e = e_{maximum}$

If, $f > 0.15$, then fix $f = 0.15$

Step-3: Now take $f = 0.15$, $e = e_{maximum}$ and find the actual velocity will be provided on the highway

$$\Rightarrow \quad e + f = \frac{V_a^2}{127R}$$

$$\Rightarrow \quad e_{max} + 0.15 = \frac{V_a^2}{127R}$$

If $V_{Design} < V_{actual}$, then O.K.

If $V_{Design} > V_{actual}$, then restrict the speed by providing speed limits sign.

Example-3.6 The radius of a horizontal circular curve is 150 m. The design speed is 60 kmph and the design coefficient of lateral friction is 0.15.

(*i*) Calculate the superelevation required if full lateral friction is assumed to develop.

(*ii*) Calculate the coefficient of friction needed if no superelevation is provided.

(*iii*) Calculate the equilibrium superelevation if the pressure on inner and outer wheels should be equal.

Solution:

(*i*) Superelevation is given by the relation

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

Here $f = 0.15$; $V = 60$ kmph or $v = \frac{60}{3.6}$ m/s $= 16.67$ m/s

$R = 150$ m

$$e + 0.15 = \frac{60^2}{127 \times 150} = 0.189 = 0.197 - 0.15 = 0.039$$

i.e. superelevation rate is 1 in 25.6.

(*ii*) If no superelevation is provided, $e = 0$ then coefficient of friction developed,

$$f = \frac{V^2}{127R} = \frac{60^2}{127 \times 150} = 0.189$$

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

$$e = \frac{V^2}{127R} = \frac{60^2}{127 \times 150} = 0.189 \text{ or } 1 \text{ in } 5.3$$

i.e., equilibrium superelevation rate is 1 in 5.3. However this rate of superelevation being very high, cannot be provided.

3.6.5 Radii of Horizontal Curve

Radius of curve is an important design element which decides the maximum comfortable speed of vehicle on a horizontal curve. It is impossible to design the curve with maximum superelevation and coefficient of friction because if the design speed increases in future, then complete realignment would be required.

A ruling minimum radius R_{ruling} is calculated on the basis of maximum super elevation and coefficient of friction. The radius of curve should be higher than R_{ruling}.

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

NOTE: Generally $e_{maximum} = 7\%$ and $f = 0.15$ otherwise given in the problem.

IRC recommends the value of ruling minimum and absolute value of radii of horizontal curve on the basis of design speeds.

Table-3.21 : Minimum radii of horizontal curves for different terrain conditions (in m)

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

3.6.6 Extrawidening

Additional width of carriageway that is required on horizontal curve is referred as extrawidening. The rear wheels follow the inner path on the curve as compared with the front wheels. This phenomena is called off tracking.

Reasons to provide extrawidening are:

(*i*) to avoid offtracking due to rigidity of wheel base
(*ii*) to counteract transverse skidding
(*iii*) to increase the visibility at curves
(*iv*) to encounter psychological tendency while overtaking operation

Extrawidening is split up into two parts:

(i) Mechanical widening: It is provided due to the rigidity of wheel base, when a vehicle travels on a horizontal curve, only front wheel can be controlled and the rear wheels does not follow the same path as front wheel.

Let,
R_1 = Radius of path traversed by the outer rear wheel (in m)
R_2 = Radius of path traversed by the outer front wheel (in m)
l = Length of wheel base (in m)
W_m = Mechanical widening (in m)

$$R_2^2 = R_1^2 + l^2$$

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

$$R_2^2 = R_2^2 - 2R_2W_m + W_m^2 + l^2$$

$$2R_2W_m - W_m^2 = l^2$$

$$W_m = \frac{l^2}{2R_2 - W_m} \simeq \frac{l^2}{2R} \quad \text{(where , R is mean radius of curve)}$$

If a road has n lanes, then the mechanical extrawidening is

$$W_m = \frac{nl^2}{2R}$$

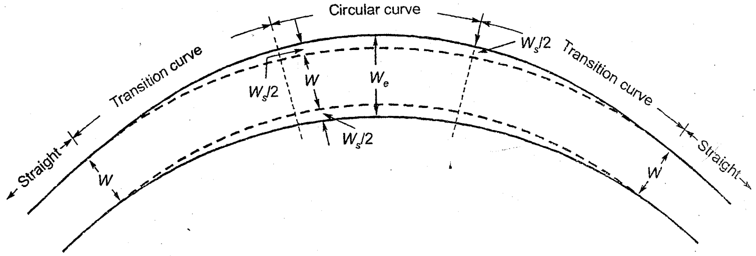


Figure-3.30 : *Extra widening of pavement on horizontal curve*

(ii) Psychological widening: There is a tendency for the drivers to drive closer to the edges of the pavements on curves. So psychological widening is also required to provide. IRC proposed an empirical formula for the psychological widening,

$$W_{ps} = \frac{V}{9.5\sqrt{R}} \text{ m} \quad \text{(where } V \text{ is in kmph)}$$

Therefore, the total extrawidening (W_e) needed to be provided on a horizontal curve will be given as

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

$$\therefore \quad W_e = \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}} \text{ m}$$

Remember IRC recommended values of extra widening for single and two lane pavement are given below;
(*i*) If $R > 300$ m, then extrawidening is not provided.
(*ii*) If $R < 50$ m , then extrawidening is provided at inner edge
(*iii*) If $50 < R < 300$ m, then extrawidening is provided at both the edges.

3.6.7 Curve Resistance for Turning Vehicle

When a vehicle turns on a horizontal curve by turning the front wheels, but the rear wheels do not turn as shown in Figure 3.31. The direction of rotation of rear and front wheels are different. So there is some loss in the tractive force.

Thus the loss of tractive force due to turning of a vehicle on a horizontal curve is termed as curve resistance,

$$\text{Curve Resistance} = T - T\cos\alpha$$

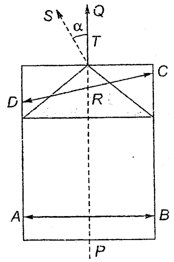


Figure-3.31 : *Curve resistance for turning vehicle*

3.6.8 Grade Compensation at Curves on Hill Roads

When there is a horizontal curve in addition to gradient then there will be increased resistance to traction due to both curve and gradient. Therefore it is necessary to compensate the gradient at horizontal curve.

Grade compensation as taken is minimum of $\frac{30+R}{R}\%$ or $\frac{75}{R}\%$, where 'R' is the radius of curve in metres.

$$\text{Compensated gradient} = \text{Gradient} - \text{Grade compensation}$$

NOTE: According to IRC grade compensation is not necessary for the gradient flatter than 4%.

Example-3.7 While aligning a hill road with a gradient of 7%, a horizontal curve of radius 80 m is provided. Find the compensated grade.

Solution:

Grade compensation,

$$\text{Minimum of} = \frac{30+R}{R}\% \text{ or } \frac{75}{R}\%$$

$$\text{G.C.} = \frac{30+80}{80} \text{ or } \frac{75}{80} = 1.375 \text{ or } 0.9375 = 0.9375\%$$

$$\text{Compensated grade} = 7 - 0.9375 = 6.0625\%$$

3.7 Transition Curves

When a vehicle traveling on a straight road enters into a horizontal curve instantaneously, it will cause discomfort to the driver. To avoid this, it is required to provide a transition curve. This may be provided either between a tangent and a circular curve or between two branches of a compound or reverse curve.

The objectives of providing a transition curves are:

(*i*) To gradually introduce the centrifugal force between straight and circular curve.
(*ii*) To avoid the certain jerk.
(*iii*) To gradual introduction of superelevation and extra widening.
(*iv*) To enable the driver turn the steering gradually for comfort and security.
(*v*) To improve aesthetic appearance.

A transition curve should satisfy the following conditions:

(*i*) It should meet the straight path tangentially.
(*ii*) It should meet the circular curve tangentially.
(*iii*) It should have the same radius as that of the circular curve at its junction with the circular curve.
(*iv*) The rate of increase of curvature and superelevation should be the same.

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

(*i*) Radius of circular curve, R
(*ii*) Design speed, V
(*iii*) Allowable rate of change of centrifugal acceleration, C
(*iv*) Maximum amount of super elevation, E which depends on the maximum rate of superelevation, e and the total width of the pavement, B at the horizontal curve
(*v*) Allowable rate of introduction of superelevation
(*vi*) Rotation of pavement cross-section either about the inner edge or the centre line

3.7.1 Different Types of Transition Curves

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

(*i*) Spiral or Clothoid
(*ii*) Bernoulli's Lemniscate
(*iii*) Cubic Parabola

The general shapes of these three curves are shown in Figure 3.32. 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 increases.

NOTE: According to IRC ideal shape for transition curve is spiral because rate of change of radial acceleration (C) remains constant.

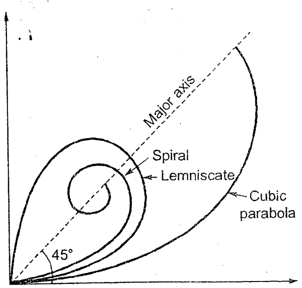


***Figure-3.32 :** Types of transition curves*

3.7.2 Length of Transition Curve

The length of transition curve is designed to fulfill three conditions i.e.,

(*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, and
(*iii*) minimum length by IRC empirical formula

The length of transition curve fulfilling all the three conditions (or the highest of the three values) is generally accepted.

Length of Transition Curve by the Rate of Change of Radial Acceleration

Radial acceleration at any point on a circular curve is $\left(\frac{V^2}{R}\right)$ m/s^2

where, V = Velocity of the vehicle in m/s
R = Radius of the curve in m

In order to have a gradual change of radial acceleration so as not to cause discomfort to the drivers, the curvature, $\left(\frac{1}{R}\right)$, must change at a definite rate from zero to a designed value. At the tangent point the radial acceleration $\left(\frac{V^2}{R}\right)$ is zero and as the radius R is infinity.

At the end of transition curve the radius 'R' has the minimum value of 'R_m'. Hence the rate of change of radial acceleration 'C' is given by

$$C = \frac{\frac{V^2}{R} - 0}{t} = \frac{V^2}{Rt} \qquad \ldots(i)$$

Let the length of transition curve is 'L_s' (in m) and 't' is the time taken by vehicle in second to travel the transition length at uniform speed of 'v' m/s.

$$\therefore \qquad t = \frac{L_s}{v} \qquad \ldots(ii)$$

From equation (*i*) and (*ii*), we get, $C = \frac{v^2}{R\frac{L_s}{v}} = \frac{v^3}{RL_s}$

$\therefore$ The length of the transition curve is,

$$L_s = \frac{v^3}{CR} = \frac{0.0215\,V^3}{CR} \qquad \text{(Here, } V \text{ is in kmph)}$$

Remember

The rate of change of centrifugal acceleration is given by an empirical formula recommended by IRC

$$C = \frac{80}{75+V} \text{ m}^3\text{/s} \qquad [V = \text{velocity in kmph}]$$

Subject to a maximum of 0.8 and minimum of 0.5.

Length of Transition Curve by an Arbitrary Rate of Change of Superelevation

The length of transition curve can be such that the superelevation (e) is applied at a uniform rate of 1 in N. The length of transition curve 'L_s' is given by

(i) When pavement is rotated about inner edge

$$L_s = eN(W + W_e)$$

where,

$$\frac{1}{N} = \text{Rate of change of superelevation}$$

N = 150 (Plain and Rolling terrain)

N = 60 (Hilly Area)

W = Width of pavement

W_e = Extra widening

(ii) When pavement is rotated about centre

$$L_s = eN\left(\frac{W + W_e}{2}\right)$$

Empirical Formula for the Length of Transition Curve Recommended by IRC

For plain and rolling terrain, $L_s = 2.7\frac{V^2}{R}$ m

For mountainous and steep terrain, $L_s = \frac{V^2}{R}$ m

3.7.3 Design Steps of Horizontal Transition Curve Length

The design steps for the horizontal transition curve length are given below:

(i) Find the length of transition curve based on allowable rate of change of the radial acceleration

(ii) Find the length of transition curve based on allowable rate of change of superelevation

(iii) Determine the minimum required value of 'L_s' as per the empirical formula

(iv) Adopt the highest value of 'L_s' given by (i), (ii) and (iii) above as the design length of transition curve

The minimum length of transition curve for various values of radius of curve and design speeds recommended by the IRC for plain and rolling terrain and also for mountainous and steep terrain are given in Table-3.22.

3.7.4 Setting out of Transition Curve

When a transition curve is introduced between a straight and circular curve, then circular curve has to be shifted so that the transition curve meets the circular curve tangentially. The shift (S) of a circular curve is given by

$$S = \frac{L_s^2}{24R} \qquad (L_s = \text{Length of transition curve})$$

***Table-3.22**: Minimum transition length for different speeds and curve radii*

Plan and Rolling terrain							Mountainous and Steep terrain					
Curve radius (m)	Design speed (kmph)						Curve radius (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	-	-	-		-						

Example 3.8 A two lane national highway without raised kerb is passing through a plain terrain. Calculate the length of transition curve, if the radius of the circular curve is equal to the minimum ruling radius. Assume data as per IRC recommendations.

Solution:

As per IRC, assume the following data for a national highway passing through a plain terrain:

Design speed V = 100 kmph

Maximum permissible superelevation = 0.07

Rate of superelevation = 1 in 150

Maximum coefficient of friction, f = 0.15

Wheel base of largest truck, L = 6.1 m

Width for pavement without raised kerbs on straight stretches, W = 7.0 m

(i) Radius of curve = Ruling minimum radius

$$= \frac{V^2}{127(e+f)} = \frac{V^2}{127(0.07+0.15)} = \frac{V^2}{27.94} = \frac{(100)^2}{27.94} = 357.91\text{ m}$$

(*ii*) Superelevation, for 75% design speed, neglecting lateral friction

$$e = \frac{(0.75V)^2}{127R} = \frac{(0.75\times100)^2}{127\times360} = 0.123 \text{ or } 12.3\%$$

Since this value is greater than the maximum permissible value of $e = 0.07$, hence adopt $e = 0.07$ and check for coefficient of friction

$$f = \frac{V^2}{127R} - e = \frac{(100)^2}{127\times360} - 0.07 = 0.22 - 0.07 = 0.15$$

Since the value of coefficient of lateral friction 'f' equals the maximum permissible value of 0.15, the design speed of 100 kmph with superelevation $e = 0.07$ is safe.

(*iii*) Since the radius of the horizontal curve is greater than 300 m, no extrawidening is required at the curves.

(*iv*) Length of transition curve

(a) By rate of change of centrifugal acceleration

$$C = \frac{80}{75+V} = \frac{80}{75+100} = 0.457 \text{ m/s}^3 \not< 0.5 \text{ m/s}^3$$

$$\therefore \quad C = 0.5 \text{ m/s}^3$$

$$L_s = \frac{0.0215\times(100)^3}{0.5\times360} = 119.44 \text{ m say } 120 \text{ m}$$

(b) By the rate of change of superelevation

Assuming the road to be rotated about the central line

$$L_s = \frac{7.0}{2}\times0.07\times150 = 36.75 \text{ m}$$

$\therefore$ Adopt transition length, $L_s = 120$ m

Example-3.9 Calculate the length of transition curve on a plain terrain 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:

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

Allowable rate of change of centrifugal acceleration as per equation

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

This value is between 0.5 and 0.8 and hence accepted

$$L_s = \frac{0.0215\,V^3}{CR} = \frac{0.0215\times65^3}{0.57\times220} = 47.1 \text{m}$$

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

Superelevation rate,
$$e = \frac{V^2}{127R} = \frac{65^2}{127\times220} = 0.15$$

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

$$f = \frac{V^2}{127R} - e = \frac{65^2}{127\times220} - 0.07 = 0.15 - 0.07 = 0.08$$

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

$$= \frac{E}{2} = \frac{eB}{2} = \frac{0.07\times7.5}{2} = 0.26 \text{ m}$$

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

$$L_s = \frac{EN}{2} = 0.26\times150 = 39 \text{ m}$$

(*iii*) Minimum value of L_s as per IRC $= \frac{2.7V^2}{R} = \frac{2.7\times65^2}{220} = 51.9$ m

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

$$\text{Shift } S = \frac{L_s^2}{24R} = \frac{52^2}{24\times220} = 0.51 \text{ m}$$

3.8 Set Back Distance

Set back distance is the clear distance required from centre line of the curve to the obstruction on the innerside of the curve to provide adequate sight distances as shown in **Figure 3.33**.

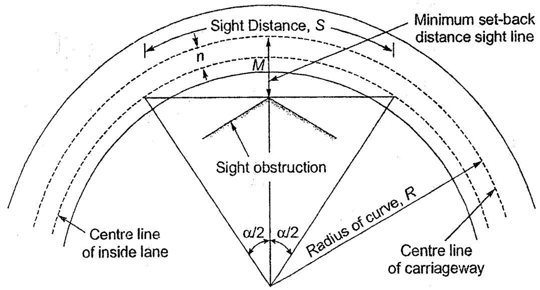


***Figure-3.33 :** Visibility at horizontal Curves*

There are two cases in the calculation of setback distance:

Case-1: When the length of a horizontal curve is greater than the sight distance ($L_C > SD$)

(a) For Single Lane:

Let, M is the setback distance and sight distance (SD) is measured along the centre line, then the setback distance from the centre line of road is given by

$$M = \left(R - R\cos\frac{\alpha}{2}\right)\text{m}$$

As we know that if 'R' is the radius of a circle then the length of an arc 'l' subtending an angle of 'α' degree at the centre is given by

$$l = R\alpha$$

Here, length of arc = SD

$\therefore$ $R\alpha = SD$

On dividing the above equation by '2', we get

$$R\frac{\alpha}{2} = \frac{SD}{2} \Rightarrow \frac{\alpha}{2} = \frac{SD}{2R}\text{ radian}$$

As 'α' is in degree so, $\frac{\alpha}{2} = \frac{SD}{2R} \times \frac{180°}{\pi}$

(b) For Multi Lane:

Let $2d$ is the total width of a lane, then setback distance from the centre line of road having only two lane is given as

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

Similarly for a road of 'n' lanes, setback distance is given by

$$M = R - (R - (n-1)\,d)\cos\frac{\alpha}{2}$$

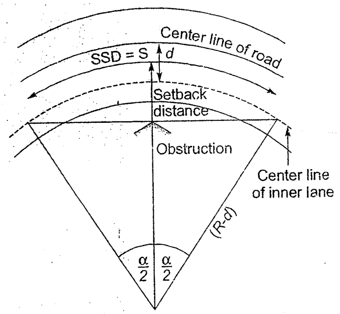


Figure-3.34 : Set back distance for multilane

Now if we have to calculate the setback distance from the centre line of inner lane, then it will be given as

For a two lane road, $M = (R-d) - (R-d)\cos\frac{\alpha}{2}$

and, $\frac{\alpha}{2} = \frac{SD}{2(R-d)} \times \frac{180}{\pi}$

For a 'n' lane road, $M = (R-(n-1)d) - (R-(n-1)d)\cos\frac{\alpha}{2}$

and, $\frac{\alpha}{2} = \frac{SD}{2(R-(n-1)\,d)} \times \frac{180}{\pi}$

Case-2 : When the length of a horizontal curve is less then the sight distance ($L_C < SD$).

(a) For Single Lane:

Figure 3.35 shows the setback distance from the centre line of the road i.e. AC.

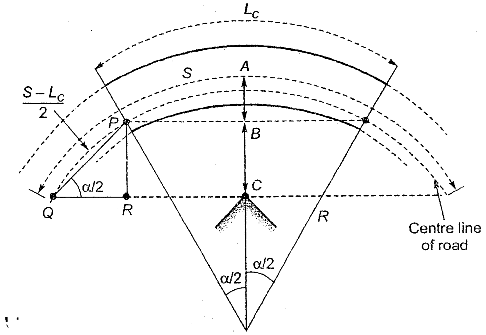


***Figure-3.35 :** Setback distance when $L_C < SD$*

In this case the setback distance is divided into two parts i.e. M_{AB} and M_{BC}

$$\therefore \quad M = M_{AB} + M_{BC}$$

Now, 'M_{AB}' value is derived similarly as of the previous case

$$\therefore \quad M_{AB} = R - R\cos\frac{\alpha}{2}$$

But in case of 'M_{BC}', we consider a triangle PQR, so

$$PR = M_{BC},\ \angle Q = \frac{\alpha}{2} \text{ and } PQ = \frac{S - L_C}{2}$$

$$\therefore \quad PR = M_{BC} = \left(\frac{S - L_C}{2}\right)\sin\frac{\alpha}{2}$$

Hence, the setback distance from the centre line of road is

$$M = \left(R - R\cos\frac{\alpha}{2}\right) + \left(\frac{S - L_C}{2}\right)\sin\frac{\alpha}{2} \text{ and } \frac{\alpha}{2} = \frac{L_C}{2R} \times \frac{180°}{\pi}$$

(b) For Two Lane:

Setback distance from the centre line of the road is shown in the Figure 3.36.

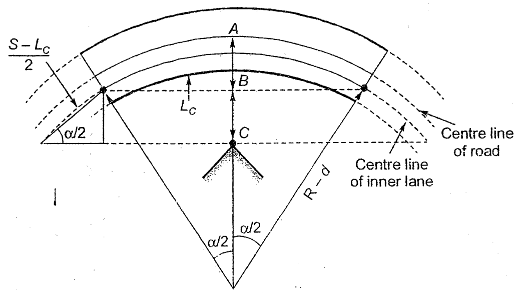


***Figure-3.36 :** Setback distance when $L_C < SD$*

Setback distance is given as,

$$M = M_{AB} + M_{BC}$$

$$\therefore \quad M = R - (R-d)\cos\frac{\alpha}{2} + \left(\frac{S-L_C}{2}\right)\sin\frac{\alpha}{2}$$

and
$$\frac{\alpha}{2} = \frac{L_C}{2(R-d)} \times \frac{180^\circ}{\pi}$$

Setback distance from centre line of inner lane is

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

and
$$\frac{\alpha}{2} = \frac{180 \times L_c}{2\pi(R-d)}$$

Example-3.10 There is a horizontal highway curve of radius 400 m and length 200 m on this highway. Compute the setback distances required from the centre line of the curve so as to provide for

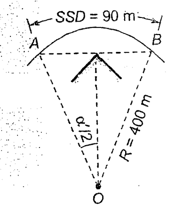
(*i*) Stopping sight distance of 90 m

(*ii*) 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:

(*i*) The stopping sight distance (SSD) of 90 m is less than the circular curve length of 200 m.



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

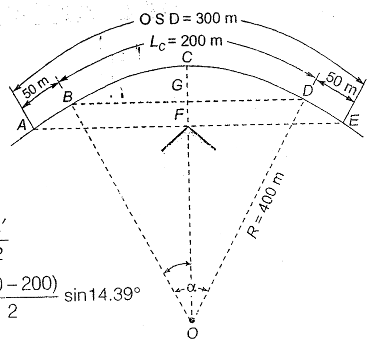
$$m = R - (R-d)\cos\frac{\alpha'}{2}$$

$$= 400 - (400 - 1.9)\cos 6°29' = 4.45 \text{ m}$$

(*ii*) The overtaking sight distance of 300 m is greater than circular curve length which is 200 m. Therefore the required setback distance is $CF = CG + GF$ and is given by equation

$S = 300$ m, $L_C = 200$ m

$R = 400$ m, $d = 1.9$ m



$$\frac{\alpha'}{2} = \frac{180\, L_C}{2\pi(R-D)}$$

$$= \frac{180 \times 200}{2\pi(400-1.9)} = 14.39°$$

Setback 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 setback 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.

3.9 Vertical Alignment

Vertical Alignment of a road consists of vertical curves and gradients.

Gradient

Gradient is the rate of rise or fall along the length of road with respect to horizontal. It is expressed as 1 in n (1 vertical to n horizontal) or n% i.e. n in 100.

There are different types of gradient:

(i) Ruling Gradient: It is the maximum gradient within which designer wants to design the vertical profile of the road, hence it is also known as design gradients. It depends upon topography, length of the grade, design speed, pulling power of vehicle and presence of horizontal curves.

(ii) Limiting Gradient: It is steeper than ruling gradient and it is provided only when there is an enormous increase of cost of construction with ruling gradient. On rolling and hilly terrain, limiting gradient may be frequently adopted but the length of limiting gradient stretch should be restricted.

(iii) Exceptional Gradient: It is steeper than ruling gradient and limiting gradient and provided only, if the situation is unavoidable. Length of exceptional gradient stitch should not be more than 100 m.

(iv) Minimum Gradient: It is provided along the length of road for drainage point of view. Minimum gradient for cement concrete drain is 1 in 500.

Table-3.23 : Gradients for different terrain types - Indian practice

S.No.	Terrain type	Rulling Gradient (%)	Limiting Gradient (%)	Exceptional Gradient (%)
1.	Plain	3.3 (1 in 30)	3.3 (1 in 20)	6.7 (1 in 15) (for short distance not exceeding 100 m at a stretch)
2.	Rolling	3.3 (1 in 30)	5 (1 in 20)	6.7 (1 in 15) (for short distance not exceeding 100 m at a stretch)
3.	Mountainous	5 (1 in 20)	6 (1 in 16.7)	7 (1 in 14.3) (for a distance not more than 100 m at a stretch)
4.	Steep			
	(i) Upto 3000m height above mean sea level.	3.3 (1 in 30)	6 (1 in 16.7)	7 (1 in 14.3) (for a distance not more than 100 m at a stretch)
	(ii) Above 3000m height above mean sea level.	6 (1 in 16.7)	7 (1 in 14.3)	8 (1 in 12.5) (for a distance not more than 100 m at a stretch)

NOTE

Critical Length of Grade : The maximum length of ascending gradient covered by a loaded truck without undue reduction in speed is called critical length of grade for a design. While reduction in speed of about 25 kmph may be considered reasonable limit.

Vertical Curve

Vertical curves are provided at the intersections of different grades to smoothen the vertical profile. The disadvantages of not providing vertical curves are:

(*i*) Accident due to inadequate visibility on a high and blind culvert over which roads profile has not been properly designed as a vertical curve.

(*ii*) Discomfort and damage due to humps and trough, although these may not be a source of accidents but they make traveling very uncomfortable.

(*iii*) It is much cheaper and easier to construct good curves in the first instance than to set right road curves later on.

The minimum length of vertical curve as per the IRC recommendations are given Table 3.24.

Table-3.24 : Minimum length of vertical curves

Design speed (kmph)	Maximum grade change (%) not requiring a vertical curve	Maximum length of vertical curve in m (for higher grade change)
35	1.5	15
40	1.2	20
50	1.1	30
65	0.8	40
80	0.6	50
100	0.5	60

The vertical curves used in the highway are of two types

(*i*) Summit curve (*ii*) Valley curve

3.9.1 Summit Curve

A summit curve is a vertical curve with convexity upward or concavity downward. This occur when an ascending gradient intersects a descending gradient or when an ascending gradient meets another ascending gradient or an ascending gradient meets a horizontal or when a descending gradient meets an another descending gradient as shown in Figure 3.37.

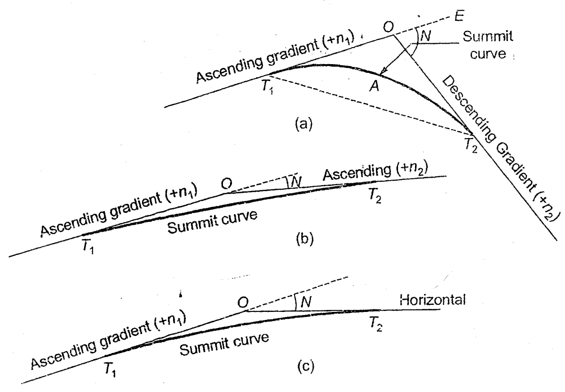


Figure-3.37 : *Summit curve, showing deflection angle*

Square parabola is the best shape for summit curve due to good riding qualities, simplicity of calculation work and uniform rate of change of grade. Ideal shape for summit curve is circular because sight distance available throughout the curve is constant.

Generally there is no problem of discomfort on summit curve because gravity force acts downwards and centrifugal force acts upwards, part of a pressure in the tyre is released. Length of summit curve depends only on sight distance (SSD, ISD and OSD).

Deviation Angle of Summit Curve

Deviation angle of a curve is expressed by the algebraic difference of the grade angles. If n_1 and n_2 are the grade angles for the two curves, then assigning proper signs (i.e., + for ascending and – for descending), the deviation angle N for the Figure 3.37 (a) is given by

$$N = \text{Angle } EOT_2 = (+n_1) - (-n_2) = n_1 + n_2$$

Thus deviation angle of a summit curve is the angle which measures the change of direction in the path of motion at the intersection of two grade lines.

Do yourself : Find out the deviation angle of a curve -formed at the intersection of an ascending gradient of 3% and an another ascending gradient of 2%. **(Ans. 0.01)**

Design of Summit Curve

Let T_1O and T_2O be the two intersecting grade lines having grade angles n_1 and n_2 respectively as shown in Figure 3.38.

T_1AT_2 be the simple parabolic curve between the tangent points T_1 and T_2 and y be the intercept between the curve and the grade line T_1O at a horizontal distance x from T_1, then general equation of parabola is given by

$$y = ax^2 \quad \text{(where, } a \text{ is constant)...}(i)$$

From Figure 3.38,

$$PT_2 = PR + RT_2$$

$$= L\frac{n_1}{2} + L\frac{n_2}{2} = \frac{L}{2}(n_1 + n_2) = \frac{L}{2}N$$

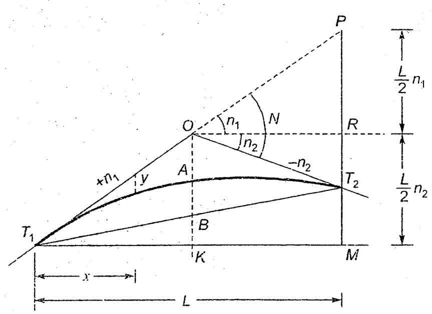


Figure-3.38 : *Parabolic summit vertical curve*

At the end point T_2 of the curve

$$x = L \text{ and } y = PT_2 = \frac{L}{2}N$$

Substituiting these values in equation (i), we get

$$\frac{LN}{2} = aL^2$$

$$\therefore \quad a = \frac{N}{2L}$$

Hence, the equation of parabola is given as

$$y = \frac{Nx^2}{2L}$$

Radius of Curvature of Summit Curve

Let (x, y) be the cartesian coordinate of any point on the curve and R, the radius of curvature at that point for a flat curve is

$$\frac{1}{R} = \frac{d^2y}{dx^2} \qquad ...(i)$$

The equation of a summit parabolic curve is

$$y = ax^2$$

$$\therefore \qquad \frac{dy}{dx} = 2ax$$

and

$$\frac{d^2y}{dx^2} = 2ax \qquad ...(ii)$$

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

$$\frac{1}{R} = 2a \quad \text{or} \quad R = \frac{1}{2a}$$

which is a constant. Thus parabolic curve is virtually a circular arc

But

$$a = \frac{N}{2L}$$

$$\therefore \qquad R = \frac{2L}{2N} = \frac{L}{N}$$

The minimum radius of the summit curve should be such that the rate of radial deceleration should not be greater than 0.75 m/s^2. Therefore in limiting case

$$\frac{(0.278\,V)^2}{R} = 0.75 \quad \text{or} \quad R = \frac{(0.278\,V)^2}{0.75}$$

The value of minimum radii for different design speeds is given in Table 3.25.

Table-3.25 : Minimum radii of summit curves for various speeds

Speed (kmph)	25	30	35	40	50	60	65	80	100
Minimum Radius (m)	65	95	130	170	250	380	440	670	1045

Length of Summit Curve

The length of summit curve depends only on sight distance i.e. *SSD*, *ISD* and *OSD*. The length of a summit curve is a function of

(*i*) the deviation angle, N (*ii*) the required sight distance

In determining the length of the summit curve, two cases have to be considered

Case-1: When the length of curve exceeds the required sight distance i.e., $L > S$.

Let 'L' is the length of curve and 'S' is the sight distance.

As we know that the equation of summit curve is

$$y = \frac{Nx^2}{2L}$$

For $x = S_1$, $y = H$ and $x = S_2$, $y = h$

$$\therefore \qquad H = \frac{NS_1^2}{2L}$$

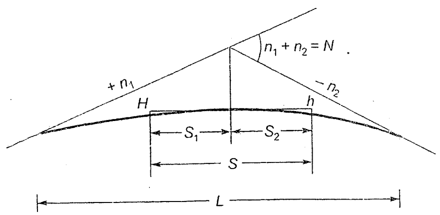


Figure-3.39 : *Summit curve when S is less than L*

and

$$h = \frac{NS_2^2}{2L}$$

$$\Rightarrow \qquad S_1 = \sqrt{\frac{2LH}{N}} \quad \text{and} \quad S_2 = \sqrt{\frac{2Lh}{N}}$$

Sight distance,

$$S = S_1 + S_2$$

$$\Rightarrow \qquad S = \sqrt{\frac{2LH}{N}} + \sqrt{\frac{2Lh}{N}} = \sqrt{\frac{2L}{N}}\left[\sqrt{H} + \sqrt{h}\right]$$

On squaring both sides, we get

$$\Rightarrow \qquad S^2 = \frac{2L}{N}\left[\sqrt{H} + \sqrt{h}\right]^2 \Rightarrow \frac{NS^2}{2L} = \left[\sqrt{h} + \sqrt{h}\right]^2$$

$$\therefore \qquad L = \frac{NS^2}{2\left[\sqrt{H} + \sqrt{h}\right]^2}$$

Where, 'H' is height of eye of driver from road surface and 'h' is height of object above pavement surface.

For SSD, $H = 1.2$ m and $h = 0.15$ m

$$\therefore \qquad L = \frac{NS^2}{4.4} \text{ m}$$

For ISD/OSD, $H = 1.2$ m and $h = 1.2$ m

$$\therefore \qquad L = \frac{NS^2}{9.6} \text{ m}$$

Case-2 : When the length of the curve is less than the required sight distance, i.e., $L < S$.

The length of summit curve is given as,

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

For SSD,

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

For ISD/OSD,

$$L = 2S - \frac{9.6}{N} \text{ m}$$

3.9.2 Valley Curve or Sag Curve

A valley curve is a vertical curve with concavity upward or convexity downward as shown in Figure 3.40.

This is formed when a descending gradient intersects an ascending gradient or when a descending gradient meets another descending gradient or when a descending gradient joins a horizontal path or when an ascending gradient meets an another ascending gradient as shown in Figure 3.41.

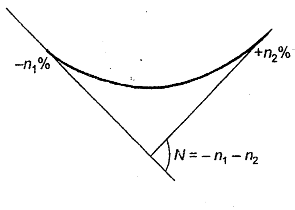


Figure-3.40 : *Valley curve*

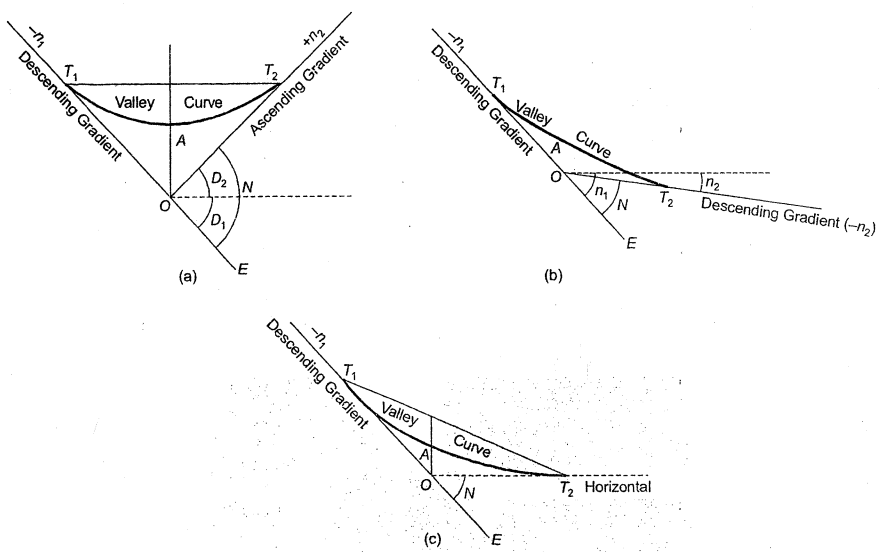


Figure-3.41 : *Valley curve, showing total deflection angle*

A valley curve is usually madeup of two transition curve of equal length without having a circular curve in between. Cubic parabola is generally preferred for valley curve to gradually introduce the centrifugal force. In valley curve, centrifugal force and gravity force both acts downwards, hence create discomfort for passengers.

Deviation Angle of Valley Curve

As in the case of summit curves, in this case also, deviation angle is the algebraic difference of the two grade angles.

NOTE: There is no restrictions to sight distance at valley curve during day time but at night only source of visibility is with the help of headlights in case of inadequate street lights.

Length of Valley Curve

Length of valley curve is designed on the basis of two criteria:

(i) Based on Comfort Condition: In this criteria the rate of change of centrifugal acceleration is limited to a comfortable zone of about 0.6 m/sec^3, then the length of transition curve is given by

$$L_s = \frac{v^3}{CR} \quad ...(i)$$

and $$R = \frac{L_s}{N} \quad ...(ii)$$

$$\therefore \quad L_s = \frac{v^3}{\frac{CL_s}{N}} \Rightarrow L_s^2 = \frac{Nv^3}{C} \Rightarrow L_s = \sqrt{\frac{Nv^3}{C}} \text{ m}$$

$\therefore$ Length of valley curve, $L_v = 2L_s = 2\sqrt{\frac{Nv^3}{C}}$ m

If $C = 0.6$ m/s^3 then $L_v = 0.38\,(Nv^3)^{1/2}$

(ii) Based on Headlight Sight Distance:

Case-1 : When the length of a valley curve is more than the head light sight distance. ($L_v >$ HSD/SSD) Let 'h' is the height of head light above road surface and 'S' is the head light sight distance which should be atleast equal to stopping sight distance.

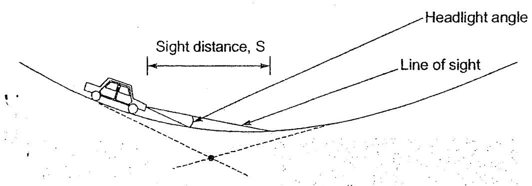


Figure-3.42 : *Headlight sight distance*

The length of valley curve, if α is the beam angle, is given by

$$L_v = \frac{NS^2}{2h + 2S\tan\alpha} \text{ m}$$

If, $h = 0.75$ m and $\alpha = 1°$

$$L_v = \frac{NS^2}{1.5 + 0.035S} \text{ m}$$

Case-2 : When the length of valley curve is less than the head light sight distance ($L_v < HSD/SSD$)

In this case, the length of valley curve is given by

$$L_v = \left(2S - \frac{2h + 2S\tan\alpha}{N}\right) = 2S - \frac{1.5 + 0.035S}{N} \text{ m}$$

Illustrative Examples

Example-3.11 How much should the outer edges of the pavement be raised with respect to the central line on a two-lane road designed to cater for mixed traffic at a speed of 65 km/hr on a horizontal curve of radius 160 m, if

(i) the pavement is rotated w.r.t. the central line, and

(ii) the pavement is rotated w.r.t. the inner edge.

Solution:

For mixed traffic conditions, superelevation should counteract the centrifugal force for 75 percent of design speed,

$$\therefore \quad e = \frac{(075\,V)^2}{127R} = \frac{V^2}{225R} = \frac{(65)^2}{225 \times 160} = 0.1174$$

Since this value is greater than the maximum permissible value of 0.07, adopt $e = 0.07$ or 7 percent.

(*i*) Raising of outer edge w.r.t. centre

Assume total width of 2 lane pavement = 7 m

$$\text{Raising of outer edge w.r.t. centre} = \frac{7}{2}\times 0.07 = 0.245 \text{ m}$$

(*ii*) Raising of outer edge w.r.t. inner edge = 7 × 0.07 = 0.49 m

Example-3.12 **Calculate the stopping sight distance for a design speed of 65 km/hr assuming the coefficient of friction as 0.36 and total reaction time of drivers as 2.5 seconds.**

Solution:

Design speed, $V = 65$ km/hr

Coefficient of friction, $f = 0.36$

Total reaction time, $t = 2.5$ seconds

Lag distance = 0.278 Vt = 0.278 × 65 × 2.5 = 45.18 m

$$\text{Braking distance} = \frac{V^2}{254f} = \frac{(65)^2}{254\times 0.36} = 46.16$$

Stopping distance = Lag distance + braking distance
= 45.18 + 46.16 = 91.34 m

Example-3.13 **A car travelling at 22.22 m/sec is overtaking another car moving at 16.67 m/sec on a two lane undivided highway. Assuming an acceleration of the overtaking car as 0.7 m/sec²; Calculate (*i*) minimum overtaking sight distance; and (*ii*) minimum and desirable length of overtaking zones.**

Solution:

Since it is a two-lane undivided highway, vehicles can overtake and there can be a vehicle coming in the opposite direction also.

Minimum overtaking sight distance = $d_1 + d_2 + d_3$

Assume design speed as the speed of overtaking vehicle

$v = 22.22$ m/sec

Also, $(v - m) = v_b = 16.67$ m/sec

$a = 0.7$ m/sec^2

$d_1 = (v_b) \times t_1$

Taking $t_1 = 2$ sec, $d_1 = 16.67 \times 2 = 33.34$ m

$S = 0.7v_b + 6 = 0.7 \times 16.67 + 6 = 11.67 + 6 = 17.67$ m

$$t = \sqrt{\frac{4\times S}{a}} = \sqrt{\frac{4\times 17.67}{0.7}} = 10.05 \text{ sec}$$

$d_2 = 2S + v_Bt = 2 \times 17.67 + 16.67 \times 10.05 = 35.34 + 166.53 = 201.87$ m

$d_3 = v \times t = 22.20 \times 10.05 = 223.11$ m

Minimum overtaking sight distance = $d_1 + d_2 + d_3$ = 33.34 + 201.87 + 223.11 = 458.32 m

Say 460 m. Minimum Length of overtaking zone

= 3 × minimum overtaking sight distance

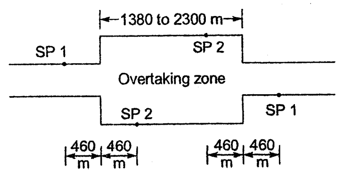
= 3 × 460 = 1380 m

Desirable length of overtaking zone

= 5 × minimum overtaking sight distance

= 5 × 460 = 2300 m

The overtaking zone is shown in figure.



Example-3.14 **A falling gradient of 1 in 20 meets a rising gradient of 1 in 40 on a Major District Highway in plain country. Find the length of the valley curve, which should provide safe driving at night.**

Solution:

$$\text{Deviation angle,}\quad N = \left|-\frac{1}{20} - \left(\frac{1}{40}\right)\right| = |-0.075| = 0.075$$

For a Major District Highway in plain country

Design speed = 80 km/hr; and Stopping distance = 120 m

(*i*) Length of valley curve from comfort condition

$= 0.38\,(NV^3)^{1/2} = 0.38\,\{0.075 \times 80^3\}^{1/2}$

$= 0.38 \times 195.96 = 74.46$ m (say 75 m)

(*ii*) Assuming the length of the valley curve to be greater than the stopping distance i.e., $L > S$.

$$L = \frac{NS^2}{1.5+0.035S} = \frac{0.075\times(120)^2}{1.5+0.035\times 120}$$

$$= \frac{0.075\times 120^2}{1.5+4.2} = \frac{0.075\times 120^2}{1.5+4.2}$$

$$= \frac{0.075\times 120^2}{5.7} = 189.47 \text{ (say 190 m)}$$

Since this value is greater than stopping distance of 120 m, hence the assumption is correct and 190 m length of valley curve is essential for safe driving at night.

From figure also the length of the valley curve is 190 m for $N = 0.075$ and $v = 80$ km/hr.

Hence adopt $L = 190$ m

(*iii*) Check for impact,

$$\text{Impact factor} = \frac{1.59NV^2}{L} = \frac{1.59\times 0.075\times 80^2}{190} = 4.02$$

Which is less than the maximum allowable impact factor of 17.

Hence 190 m length of the valley curve is adequate.

Example-3.15 **A valley curve is formed by a descending gradient of 1/25 and an ascending gradient of 1/30. Design the length of valley curve to fulfill both comfort condition and headlight sight distance.**

Design Speed = 80 kmph

$f_{longitudinal} = 0.35$, $f_{lateral} = 0.15$

Solution:

$$v = 80\text{ kmph} \quad \text{or} \quad 22.22\text{ m/s}$$

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

$$SSD = 22.22 \times 2.5 + 22.22 \times 2.5 + \frac{(22.22)^2}{2 \times 9.81 \times 0.35} = 127.45$$

$$N = \left| -\frac{1}{25} - \left(\frac{1}{30}\right) \right| = 0.0733$$

Let length of valley curve is L_v

(*i*) Comfort condition, $L_v = 2\sqrt{\frac{Nv^3}{C}} = 2\sqrt{\frac{0.0733 \times (22.22)^3}{0.6}} = 73.14$

(*ii*) Headlight Sight Distance (HSD)

Let, $L_v >$ HSD/SSD

$$L_v = \frac{NS^2}{2h + 2s\tan\alpha} = \frac{NS^2}{1.5 + 0.035S} = \frac{0.0733 \times (127.45)^2}{1.5 + 0.035 \times 127.45}$$

$= 199.75$ m > (SSD = 127.45 m)

So, length of valley curve is maximum of both the condition, $L_v = 199.75$ m

Example-3.16 **If ruling gradient is 1 in 20. What will be the grade compensation and compensated gradient for a curve of radius 150 m.**

Solution:

Data, given Ruling gradient $= \frac{1}{20}$, Radius of curve = 120 m

$$\text{Grade compensation} = \left(\frac{30+R}{R}\right)\% \text{ or } \left(\frac{75}{R}\right)\%$$

$$= \frac{(30+150)}{150}\% \text{ or } \left(\frac{75}{150}\right)\% = 1.2\% \text{ or } 0.5\%$$

$$\text{Compensated gradient} = \left(\frac{1}{20} - \frac{0.5}{100}\right) = \frac{1}{22.22}$$

Example-3.17 **A two lane national highway having design speed of 50 kmph passing through a hilly terrain has a horizontal curve of radius equal to ruling minimum radius. Design all geometrical features of the curve. Calculate set back distance for SSD also.**

Solution:

(*i*) Ruling min radius, $R_{min} = \frac{(0.278 \times V)^2}{g(e+f)} = \frac{(0.278 \times 50)^2}{9.81 \times (0.10 + 0.15)} = 78.78\text{ m} \simeq 79\text{ m}$

(*ii*) For superelevation neglecting the lateral friction coefficient superelevation should fully counteract the centrifugal force for 75% of design speed.

$$e = \frac{(0.278 \times 0.75 \times V)^2}{(g+R)} = \frac{(0.278 \times 0.75 \times 50)^2}{9.81 \times 79} = 0.140 > 0.10$$

Therefore provide maximum superelevation for hilly region $e = 0.10$

If, $f < 0.15$

Check: $f = \frac{(0.278 \times V)^2}{gR} - e = \frac{(0.278 \times 50)^2}{(9.81 \times 79)} - 0.10 = 0.149 < 0.15$ (OK)

(*iii*) Extra widening, $W_e = \left(\frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}}\right) = \frac{2 \times (6)^2}{2 \times 79} + \frac{50}{9.5 \times \sqrt{79}} = 1.047 \simeq 1.05$ m

$\therefore$ Total width of two lane highway $= (W + W_e) = 7.0 + 1.05 = 8.05$ m

(*iv*) Length of transition curve

(a) As per rate of change of radial acceleration

$$L = \frac{v^3}{(R \times C)} \qquad \left(C = \frac{80}{(75+50)} = 0.64\text{ m/s}^3\right)$$

$$\therefore \quad L = \frac{(0.278 \times 50)^3}{0.64 \times 79} = 53.11\text{m} \qquad ...(i)$$

(b) As per IRC, for hilly region,

Length of transition curve, $L = \left(\frac{V^2}{R}\right) = \frac{(50)^2}{79} = 31.64$ m ...(*ii*)

Therefore, provide length of transition curve = 53.11 m $\simeq$ 54 m

(*v*) Set back distance:

Set back distance is measured from center of road.

Assume, $L_c <$ SSD

For SSD without considering gradient

$$\therefore \quad SSD = 0.278\,Vt_R + \frac{(0.278 \times V)^2}{2g \times (f \pm n\%)}$$

$$= (0.278 \times 50 \times 2.5) + \frac{(0.278 \times 50)^2}{2 \times 9.81 \times (0.35 \pm 0)} = 62.9\text{ m}$$

$$\therefore \quad \frac{\alpha}{360} = \frac{SSD}{2\pi(R-d)}$$

$$\frac{\alpha}{2} = \frac{180SSD}{2\pi \times (R-d)} = \frac{180 \times 62.9}{2\pi \times \left(79 - \frac{8.05}{4}\right)} = 23.41°$$

$$\therefore \quad \text{Set back distance} = R - (R-d)\cos\left(\frac{\alpha}{2}\right) + \frac{S - L_C}{2}\sin\left(\frac{\alpha}{2}\right)$$

$$= 79 - (79 - 2.01)\cos(23.41)° + 1.94 = 10.29\text{ m}$$

Example-3.18 Calculate the shift and offsets at every 30 m of a transition curve. The transition curve of 90 m long is to be used to join the ends of a 4° circular curve within the straight and circular curve.

Solution:

$$R = \text{Radius of the curve} = \frac{1720}{4} = 430 \text{ m}$$

$$S = \text{Shift} = \frac{L^2}{24R} = \frac{90 \times 90}{24 \times 430} = 0.7848 \text{ m}$$

$$\text{Offset at 30 m} = \frac{30^3}{6 \times 90 \times 430} \times 100$$

$$= 11.63 \text{ cm} \qquad \left(\text{From offset } y = \frac{x^3}{6RL}\right) = Cx^3$$

where, $C = \dfrac{1}{6RL}$

$$\text{Offset at 60 m} = \frac{60^3}{6 \times 90 \times 430} \times 100 = \frac{1}{232200} = 93.02 \text{ cm}$$

$$\text{Offset at 90 m} = \frac{90^3}{6 \times 90 \times 430} \times 100 = 313.9 \text{ cm}$$

Example-3.19 **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)

$$C = \frac{80}{(75+V)} = \frac{80}{75+80} = 0.52 \text{ m/s}^3$$

As the value of C is between 0.5 to 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 rainfall area.

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

$$L_S = \frac{v^3}{CR} = \frac{0.0215\,V^3}{CR} = \frac{0.0215 \times 80^3}{0.52 \times 500} = 42.3 \text{ m}$$

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

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

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

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

(*iii*) Check for minimum value of L_S by equation

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

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

Example-3.20 **Calculate the extra width of the pavement required on a horizontal curve of radius 700 m on a two lane highway, the design speed being 80 kmph. Assume wheel base length of 6 m.**

Solution:

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

Mechanical Widening:

$$W_m = \frac{nl^2}{2R}$$

Here given data: $n = 2$, $l = 6$ m, $R = 700$ m

$$\therefore \quad W_m = \frac{nl^2}{2R} = \frac{2 \times (6)^2}{2 \times 700} = 0.0514 \text{ m}$$

Psychological Widening:

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

where V is design speed in kmph and R is radius of horizontal curve in m

Here, $V = 80$ kmph and $R = 700$ m

$$\therefore \quad W_{ps} = \frac{80}{9.5\sqrt{700}} = 0.3183 \text{ m}$$

Hence total widening,

$$W_e = W_m + W_{ps}$$
$$= 0.0514 + 0.3183 = 0.3697 \text{ m}$$

Example-3.21 **The radius of a horizontal circular curve is 100 m. The design speed is 50 km/h and the design coefficient of lateral friction is 0.15.**

(i) Calculate design superelevation

(ii) Calculate coefficient of friction in case of no superelevation

(iii) Calculate equilibrium superelevation

Solution:

Here, $f = 0.15$, $v = 50$ km/h, $R = 100$ m

(*i*)

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

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

$$e = 0.047$$

i.e. super elevation is 1 in 21.34

(*ii*) If no superelevation is provided, $e = 0$

$$\therefore \quad f = \frac{v^2}{127R} = \frac{50^2}{127 \times 100} = 0.197$$

(*iii*) For the pressure on inner and outer wheels to be equal, f = 0

$$\therefore \quad e = \frac{v^2}{127R} = \frac{50^2}{127 \times 100} = 0.197$$

i.e. equilibrium superelevation rate is 1 in 5.1.

Important Expressions

1. Stopping sight distance = $vt + \frac{v^2}{2g(f\eta \pm 0.01n)}$
2. Overtaking sight distance = $v_b t + v_b T + vT$
3. $S = 0.7\, v_b$ + length of vehicle
4. $T = \sqrt{\frac{4S}{a}} = \sqrt{\frac{14.4S}{A}}$
5. Impact factor = $1.59\frac{Nv^2}{L}$
6. Fundamental equation for horizontal curve design $e + f = \frac{V^2}{127R}$
7. Extra widening = $\frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}}$
8. Grade compensation = Minimum of $\frac{30+R}{R}$% or $\frac{75}{R}$%
9. Length of transition curve

   (a) $L_s = \frac{v^3}{CR}$ $\left(\text{where, } C = \frac{80}{75+V}\ \text{m/s}^3\right)$

   (b) $L_s = eN(W + W_e)$

   (c) $L_s = 2.7\frac{v^2}{R}$
10. Shift of circular curve, $S = \frac{L_s^2}{24R}$
11. Length of summit curve = $\frac{NS^2}{2[\sqrt{H}+\sqrt{h}]^2}$ $(\because L > S)$

    $= 2S - \frac{2[\sqrt{H}+\sqrt{h}]^2}{N}$ $(\because L < S)$
12. Length of value curve on the basis of comfort condition $L_v = 2L_s = 2\sqrt{\frac{NV^3}{C}}$
13. Length of valley curve on the basis of *HSD*

    (a) $L_v = \frac{NS^2}{1.5 + 0.035S}$ $(L_v > HSD)$

    (b) $L_v = 2S - \frac{1.5 + 0.035S}{N}$ $(L_v < HSD)$

Summary

- Geometric design of a highway design of a highway depends upon topography, design speed, road user and vehicle characteristics, traffic capacity etc.
- **Perception time** is the time required for transmission of the sensation received through eyes, ears and body to the brain and spinal card by the nervous system.
- Under mixed traffic conditions, the different types of vehicles need to be converted to a common unit known as Passenger Car Unit(PCU) by multiplying their number with relevant equivalency factor.
- Maximum permissible width of vehicle as per IRC is 2.44 m.
- Intermediate sight distance is equal to twice of stopping sight distance.
- Ratio of maximum centrifugal force to the weight of the vehicle is called **impact factor**.
- Maximum superelevation for plain and rolling terrain is 7%.
- Reaction time for the calculation of SSD is taken as 2.5 sec and for OSD is 2.0 sec
- According to IRC, ideal shape for transition curve is spiral.
- Clear distance required from centre line of the curve to the obstruction on the innerside of the curve is known as **setback distance**.
- A summit curve is a vertical curve with concavity downward and valley curve has concavity upward.

Objective Brain Teasers

Q.1. If the ruling gradient on any highway is 3%, the gradient provided on the curve of 300 metre radius, is

(a) 2.00% (b) 2.25%

(c) 2.50% (d) 3.00%

Q.2 If the coefficient of friction on the road surface is 0.15 and a maximum super elevation 1 in 15 is provided, the maximum speed of the vehicles on a curve of 100 m radius, is

(a) 32.44 kmph (b) 62.44 kmph

(c) 52.44 kmph (d) None of these

Q.3 If the rate of change of the superelevation along a curved portion of a 7 m wide road is 1 in 150 and the maximum superelevation allowed is 1 in

15, the maximum length of the transition curve to be provided at either end is
(a) 65 m (b) 75 m
(c) 70 m (d) 80 m

Q.4 If R is the radius of a main circular curve, Δ is the angle of deflection and a is the polar deflection angle of any point, its radial distance is

(a) $3R\sqrt{\sin\frac{\Delta}{3}.\sin2\alpha}$ (b) $3R\sqrt{\sin\frac{\Delta}{2}.\sin3\alpha}$

(c) $3R\sqrt{\sin2\Delta.\sin\alpha}$ (d) None of these

Q.5 The absolute minimum radius of horizontal curve for a design speed of 60 kmph is
(a) 130 m (b) 210 m
(c) 360 m (d) None of these

Q.6 Over taking time required for a vehicle with design speed of 50 kmph and overtaking acceleration 1.25 m/s^2 to overtake a vehicle moving at a speed of 30 kmph is
(a) 5.0 sec (b) 6.12 sec
(c) 25.48 sec (d) 30 sec

Q.7 If the length of transition curve (L) is less than the sight distance (S) then, minimum offset from the obstructing building is

(a) $\frac{L(2S-L)}{8R}$ (b) $\frac{S(2L-S)}{8R}$

(c) $\frac{L(2S+L)}{8R}$ (d) $\frac{S(2S-L)}{4R}$

Q.8 What is the super elevation for a horizontal highway curve of radius 500 m and speed 100 kmph in mixed traffic condition?
(a) 8.9% (b) 6.2%
(c) 0% (d) 7.0%

Q.9 Match **List-I (Type of curve)** with **List-II (Design factor)** and select the correct answer using the codes given below the list:

List-I
A. Summit curve
B. Sag curve
C. Horizontal curve
D. Transition curve

List-II
1. Superelevation
2. Set back distance
3. Headlight sight distance
4. Right of way
5. Passing sight distance

Codes:

	A	B	C	D
(a)	4	1	3	2
(b)	5	3	2	1
(c)	4	3	2	1
(d)	5	1	3	2

Q.10 Pick up the incorrect statement of the following:
(a) As tyre pressure and load increases, friction coefficient decrease.
(b) Smooth and wet surfaces will have less coefficient of friction
(c) Old tyres are more dependable than new ones in rainy season.
(d) For a four lane road, the width of carriage way is 14 m.

Q.11 A test car is braked when travelling at a standard speed. The deceleration developed, as measured by a decelerometer fitted on the car is 4.2 m/s^2. The skid resistance is
(a) 0.39 (b) 0.43
(c) 0.26 (d) 0.49

Q.12 A car travelling at 22.22 m/s is overtaking another car moving at 16.67 m/s on a two lane undivided 0.7 m/s^2. Desirable length of overtaking zone is 'm' is
(a) 460 (b) 606
(c) 1380 (d) 2300

Q.13 A vehicle move over a horizontal curve of 48 m radius with lateral friction of 0.1. What is the maximum velocity that can be allowed without any superelevation?
(a) 9.8 m/s (b) 6.8 m/s
(c) 4.9 m/s (d) None of these

Q.14 A horizontal curve of radius 600 m an a two lane highway with width 7 m is to be designed to later for mixed traffic at a speed of 65 kmph. The raise of outer edge of the pavement, if the pavement is rotated with respect to the inner edge
(a) 0.11 m (b) 0.22 m
(c) 0.49 m (d) 1.20 m

Q.15 The off tracking distance for a vehicle with wheel base equal to 6.5 m. While negotiating a curved path with a mean radius of 125 m is
(a) 0.61 (b) 0.17
(c) 0.23 (d) 0.56

Q.16 There is a horizontal curve of radius 360 m and length 180 m. The clearance required from the center line of the inner side of the curve, so as to provide the stopping sight distance of 80 m.
(a) 1.23 m (b) 1.78 m
(c) 2.2 m (d) None of these

Q.17 A road of carriage way 7.5 m on a horizontal curve of radius 300 m. The longest wheel base of vehicles using the road is 6.1 m. Design speed is 80 kmph. The extra widening in 'm' is
(a) 0.609 (b) 0.87
(c) 0.16 (d) None of these

Q.18 The length of transition curve on the basis of allowable rate of change of centrifugal acceleration for a design speed of 80 kmph and radius 300 m is
(a) 84.23 m (b) 70.87 m
(c) 53.33 m (d) 34.4 m

Q.19 The design speed of a highway is 80 km/hr and the radius of circular curve is 150 m in plain topography. Which one of the following is the minimum length of transition curve?
(a) 115 m (b) 85 m
(c) 64 m (d) 43 m

Q.20 A valley curve is to be designed for a NH in rolling terrain where a falling gradient of 1/25 meets a rising gradient of 1/20. The design speed is 100 kmph. Find length of valley curve (C = 0.6 m/s^3 and SSD = 180 m)
(a) 80 m (b) 360 m
(c) 380 m (d) 420 m

Q.21 Also calculate the maximum value of impact factor while traversing over the curve in the above question number 20.
(a) 2.48 (b) 3.77
(c) 13.28 (d) 17.36

Q.22 A falling gradient of 1 in 20 meets a rising gradient of 1 in 40 on a major district highway in a plain country. Design speed is 80 kmph. The length of valley curve is (SSD = 20 m and c = 0.6 m/s^3).
(a) 74 m (b) 190 m
(c) 236 m (d) None of these

Q.23 The radius of horizontal circular curve is 100 m. The design speed is 50 kmph and the design lateral friction is 0.15, then what is the equilibrium superelevation, if the pressure on inner and outer wheels are equal
(a) 0.047 (b) 0.347
(c) 2.39 (d) 0.197

Direction: Each of the next consists of two statements, one labelled as the **'Assertion (A)'** and the other as **'Reason (R)'**. You are to examine these two statements carefully and select the answers to these items using the codes given below:

Codes:
(a) Both A and R are individually true and R is the correct explanation of A
(b) Both A and R are individually true but R is not the correct explanation of A
(c) A is true but R is false
(d) A is false but R is true

Q.24 **Assertion (A):** When a sharp horizontal curve is to be introduced on a road which already has the maximum permissible gradient, then gradient should be decreased.
Reason (R): The gradient should be decreased to compensate for the loss of tractive effort due to the introduction of sharp horizontal curve on the road.

Q.25 **Assertion (A):** It is general practice to provide cant to the road surface with inner edge forming the pivot point.
Reason (R): It does not change center line levels which have already been fixed at the design stage.

Answers

1. (d)	**2.** (c)	**3.** (c)	**4.** (a)	**5.** (a)
6. (d)	**7.** (a)	**8.** (d)	**9.** (b)	**10.** (c)
11. (b)	**12.** (c)	**13.** (b)	**14.** (b)	**15.** (b)
16. (c)	**17.** (d)	**18.** (b)	**19.** (a)	**20.** (c)
21. (a)	**22.** (a)	**23.** (d)	**24.** (d)	**25.** (a)

Conventional Practice Questions

Q.1 Calculate the super elevation to be provided for a horizontal curve of radius 50 m for a design speed of 40 kmph in snow bound hilly terrain. What is the maximum super elevation that can be provided and what will be the coefficient of friction then? Is the design safe? If it is not safe, what remedy do you suggest?

Ans. $e = 0.1019$, restricted to 0.10
$\mu = 0.152$
which is greater than 0.15, hence not safe. Remedy is to increase the radius or post road sign restricting the speed.

Q.2 A National Highway of width 7.5 m located in rolling terrain has a radius of 250 m. Determine the length of the transition curve making suitable assumptions (Assumptions, V = 80 kmph, e = 0.07 max, rate of attainment of e = 1 in 150, rotation of super elevation about center).

[Ans. 85.07 m]

Q.3 A summit curve is to be provided at the intersection of two gradients, +1.5 percent and −2.0 percent. What is the length required (*i*) for stopping sight distance of 180 m and (*ii*) for overtaking sight distance of 640 m? What is the vertical distance between the point of vertical intersection (PVI) and curve in either case?

Ans. (*i*) 257.7 m and 1.14 m
(*ii*) 1493.3 m and 6.56 m

Q.4 A sag curve is to be designed where two gradients meet. The gradients are −2.0 percent and +2.5 percent. The design speed is 100 kmph. Find the length of the curve (*i*) for rider comfort and (*ii*) for headlight sight distance. The stopping sight distance is 180 m.

[Ans. (*i*) 80.19 m and (*ii*) 186.67 m]

Q.5 Calculate the setback distance from the inner edge of the curve in a four lane divided carriageway, the length of the curve being 1500 m, the stopping sight distance 250 m, the lane width being 3.5 m and the radius of the curve being 400 m.

[Ans. 14.21 m]