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ENGINEERS ACADEMY

RPSC - A.En.

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CIVIL

Rajasthan Public Service Commission (RPSC)

Volume - 9

Transportation Engineering

SIERRA INNOVATIONS

1 HIGHWAY ENGINEERING

THEORY

1.1 HISTORY OF HIGHWAY PLANNING IN INDIA

1.1.1 Jayakar Committee

In 1927 Jayakar committee for Indian road development was appointed. The major recommendations and the resulting implementations were

- Committee found that the road development of the country has become beyond the capacity of local governments and suggested that Central government should take the proper charge considering it as a matter of national interest.
- One of the recommendations was the holding of periodic road conferences to discuss about road construction and development. This paved the way for the establishment of a semi-official technical body called Indian Road Congress (IRC) in 1934.
- The committee suggested imposition of additional taxation on motor transport which includes duty on motor spirit, vehicle taxation, license fees for vehicles plying for hire. This led to the introduction of a development fund called Central road fund in 1929. This fund was intended for road development.
- A dedicated research organization should be constituted to carry out research and development work. This resulted in the formation of Central Road Research Institute (CRRI) in 1950, New Delhi.
- They gave more stress on long term planning programme, for a period of 20 years (hence called twenty year plan) that is to formulate plans and implement those plans with in the next 20 years.

1.1.2 Nagpur Road Plan 1943 - 1961

The roads were divided into four classes

1. National highways which would pass through states and places having national importance for strategic, administrative and other purposes.
2. State highways which would be the other main roads of a state.
3. District roads which would take traffic from the main roads to the interior of the district. According to the importance, some are considered as major district roads and the remaining as other district roads.
4. Village roads which would link the villages to the road system.
5. They suggested that the length of the road should be increased so as to give a road density of 16kms/100sq.km.

1.1.3 Bombay Road Plan 1961 - 1981

The highlights of the plan were

- It was the second 20 year road plan (1961-1981).
- The total road length targeted to construct was about 10 lakhs.
- Rural roads were given specific attention. Scientific methods of construction was proposed for the rural roads. The necessary technical advice to the Panchayaths should be given by State PWD's.
- They suggested that the length of the road should be increased so as to give a road density of 32kms/100sq.km.
- The construction of 1600 km of expressways was also then included in the plan.

1.1.4 Lucknow Road Plan 1981 - 2001

Some of the salient features of this plan are as given below

- This was the third 20 year road plan (1981-2001). It is also called Lucknow road plan.
- It aimed at constructing a road length of 12 lakh kilometres by the year 2001 resulting in a road density of 82kms/100 sq.km.
- The plan has set the target length of NH to be completed by the end of seventh, eighth and ninth year plan periods.
- It aims at improving the transportation facilities in villages, towns etc. such that no part of country is farther than 50 km from NH.
- One of the goals contained in the plan was that expressways should be constructed on major traffic corridors to provide speedy travel.
- Energy conservation, environmental quality of roads and road safety measures were also given due importance in this plan.

1.2 CROSS SECTIONAL ELEMENTS OF PAVEMENT

The features of the cross-section of the pavement influences the life of the pavement as well as the riding comfort and safety. Camber, kerbs and geometry of various cross-sectional elements are important aspects to be considered in this regard.

1.2.1 Pavement Unevenness

The longitudinal profile of the road pavement has to be 'even' in order to provide good riding comfort to fast moving vehicles and to minimise the vehicle operation cost. Presence of undulations on the pavement surface is called 'pavement unevenness'.

The unevenness of pavement surface is commonly measured by using a simple equipment called 'Bump Integrator' (BI), in terms of 'unevenness index', which is the cumulative measure of vertical undulations of the pavement surface recorded per unit length of the road.

1.2.2 Light Reflecting Characteristics

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

1.2.3 Friction

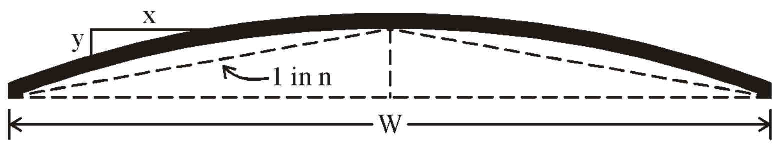
Friction between the wheel and the pavement surface is a crucial factor in the design of horizontal curves and thus the safe operating speed. Further, it also affects the acceleration and deceleration ability of vehicles. Lack of adequate friction can cause skidding or slipping of vehicles.

- Skidding happens when the path traveled along the road surface is more than the circumferential movement of the wheels due to friction.
- Slip occurs when the wheel revolves more than the corresponding longitudinal movement along the road.
- Coefficient of longitudinal friction = 0.35 to 0.4, Coefficient of lateral friction = 0.15

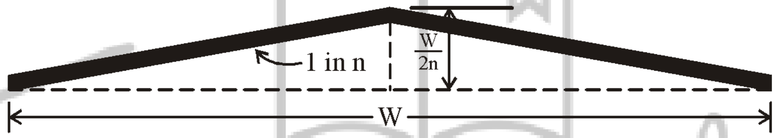
1.2.4 Camber

Camber or cant is the cross slope provided to raise middle of the road surface in the transverse direction to drain of rain water from road surface. The objectives of providing camber are

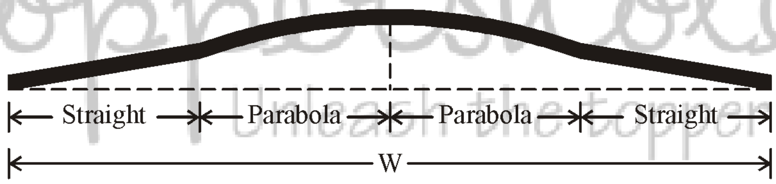
- Surface protection especially for gravel and bituminous roads.
- Sub-grade protection by proper drainage.
- Quick drying of pavement which in turn increases safety.



(a) Parabolic camber $y = \dfrac{2x^2}{nW}$



(b) Straight line camber $y = \dfrac{W}{2n}$



(c) Combination of straight and parabolic camber

Fig. : Different types of camber

Table : IRC values for camber

S. No.	Surface type	Heavy rain	Light rain
1.	Concrete/Bituminous	2 %	1.7 %
2.	Thin bituminous surface	2.5 %	2 %
3.	Gravel/WBM	3 %	2.5 %
4.	Earthen	4 %	3 %

1.2.5 Width of Carriage Way

Width of the carriage way or the width of the pavement depends on the width of the traffic lane and number of lanes. Width of a traffic lane depends on the width of the vehicle and the clearance. Side clearance improves operating speed and safety. The maximum permissible width of a vehicle is 2.44 m.

and the desirable side clearance for single lane traffic is 0.68 m. This require minimum of lane width of 3.75 m for a single lane road. However, the side clearance required is about 0.53 m, on either side and 1.06 m in the center. Therefore, a two lane road require minimum of 3.5 meter for each lane.

Table : IRC Specification for carriage way width

Traffic Lane	Width
Single lane	3.75 m
Two lane, no kerbs	7.0 m
Two lane, raised kerbs	7.5 m
Intermediate carriage	5.5 m
Multi-lane	3.5 m

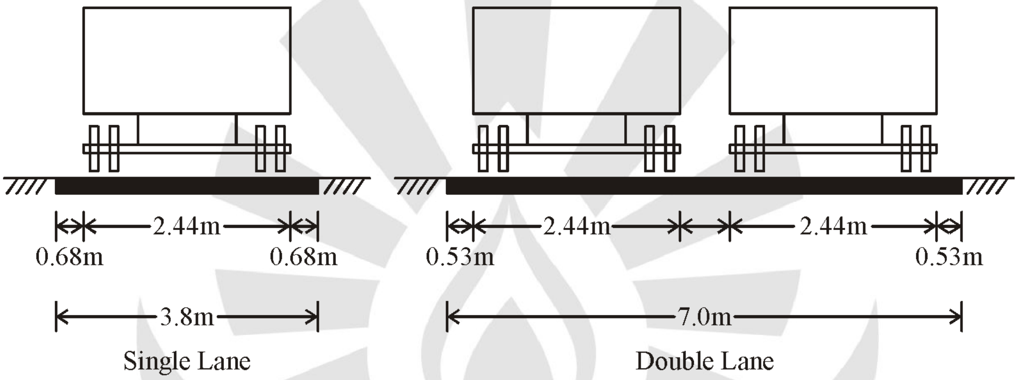


Fig. : Lane width for single and two lane roads

1.2.6 Kerbs

Kerbs indicate the boundary between the carriage way and the shoulder or islands or footpaths. Different types of kerbs are

- Low or mountable kerbs
- Semi-barrier type kerbs
- Barrier type kerbs
- Submerged kerbs

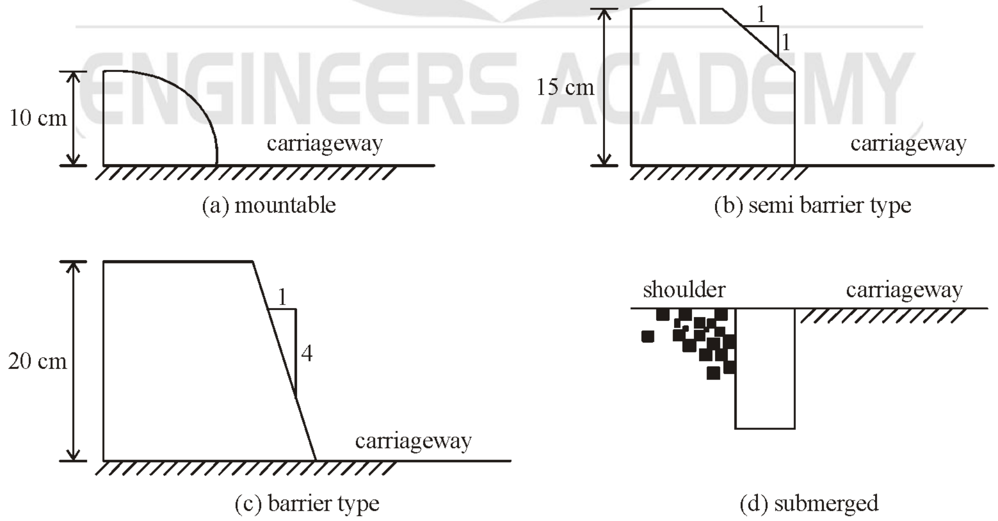


Fig. : Different types of kerbs

1.2.7 Shoulder

Shoulders are provided on both sides of the pavement all along the road in the case of undivided carriagewauy. Shoulders are provided along the outer edge of the carriageway in the case of divided carriageway. The earth shoulders should have sufficient stability to support even a loaded truck and therefore they are constructed using good quality material. In order to increase the capacity of the roadway, 'paved shoulders' are also laid on roads with high traffic flow. The minimum shoulder width recommeded by the IRC is 2.5 m.

1.2.8 Width of Formation

Width of formation or roadway width is the sum of the widths of pavements or carriage way including separators and shoulders. This does not include the extra land in formation/cutting. The values suggested by IRC are given in

Width of formation for various classed of roads

Road classification	Roadway width in m	
	Plain and rolling terrain	Mountainous and steep terrain
NH/SH	12	6.25-8.8
MDR	9	4.75
ODR	7.5-9.0	4.75
VR	7.5	4.0

1.2.9 Right of way

Right of way (ROW) or land width is the width of land acquired for the road, along its alignment. It should be adequate to accommodate all the cross-sectional elements of the highway and may reasonably provide for future development. To prevent ribbon development along highways, control lines and building lines may be provided. Control line is a line which represents the nearest limits of future uncontrolled building activity in relation to a road. Building line represents a line on either side of the road, between which and the road no building activity is permitted at all. The right of way width is governed by

- **Width of formation :** It depends on the category of the highway and width of roadway and road margins.
- **Height of embankment or depth of cutting :** It is governed by the topography and the vertical alignment.
- **Side slopes of embankment or cutting :** It depends on the height of the slope, soil type etc.
- **Drainage system :** Drainage system size which depends on rainfall, topography etc.
- **Sight distance considerations :** On curves etc. there is restriction to the visibility on the inner side of the curve due to the presence of some obstructions like building structures etc.
- **Reserve land for future widening :** Some land has to be acquired in advance anticipating future developments like widening of the road.

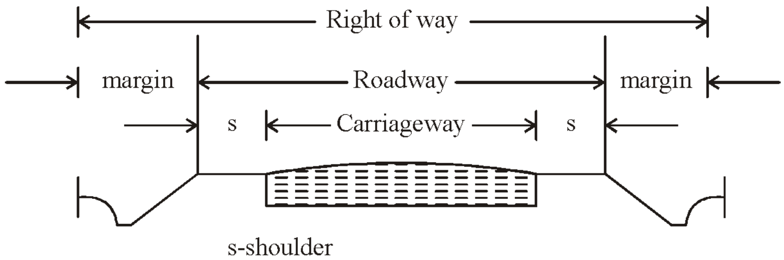


Fig. : Typical right of way (ROW)

1.2.10 Footpath

In order to provide safe facility to pedestrians to walk along the roadway, foot paths or side-walks are provided in urban areas where the pedestrain traffic is note worthy and the vehicular traffic is also heavy.

The absolute minimum width of foot path is 1.5 m and the desirable minimum width is 2.0 m; the width may be increased based on the pedestrian traffic volume.

1.2.11 Lay-byes

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

1.3 SIGHT DISTANCE

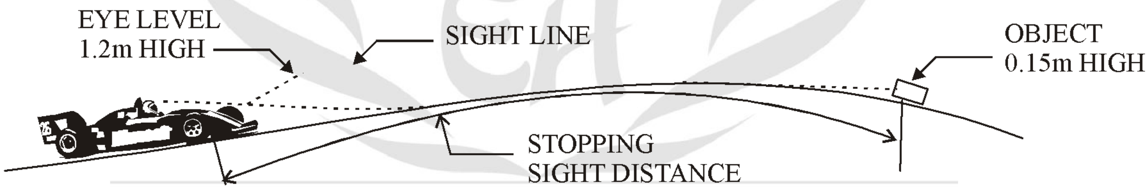
1.3.1 Types of Sight Distance

Sight distance available from a point is the actual distance along the road surface, over which a driver from a specified height above the carriage way has visibility of stationary or moving objects. Three sight distance situations are considered for design:

- Stopping sight distance (SSD) or the absolute minimum sight distance
- Intermediate sight distance (ISD) is the defined as twice SSD
- Overtaking sight distance (OSD) for safe overtaking operation
- Head light sight distance is the distance visible to a driver during night driving under the illumination of head light

1.3.2 Stopping Sight Distance

Safe sight distance to enter into an intersection



SSD = lag distance + breaking distance

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

Where v = design speed in m/sec

t = reaction time in sec

g = acceleration due to gravity

and f = coefficient of friction.

The coefficient of friction f is given below for various design speed.

Coefficient of longitudinal friction

Speed, kmph	<30	40	50	60	>80
f	0.40	0.38	0.37	0.36	0.35

- Safe sight distance to enter into an intersection with gradient n%

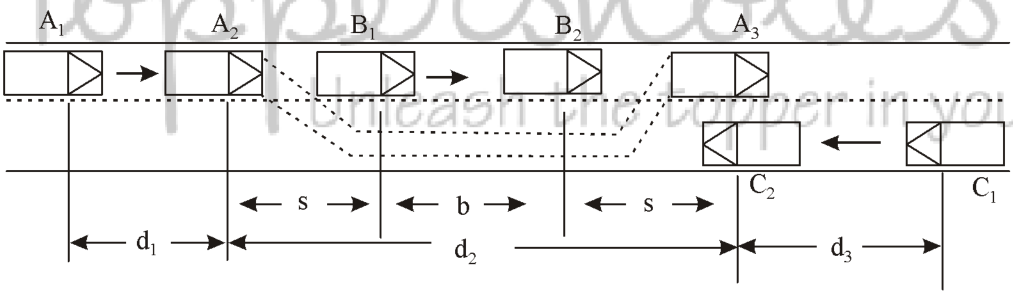
$$SSD = vt + \frac{v^2}{2g(f \pm 0.01n)}$$

where, n = gradient in %, take (+)ve sign for ascending gradient and take (–)ve sign for descending gradient.

1.3.2 Overtaking Sight Distance

The overtaking sight distance is the minimum distance open to the vision of the driver of a vehicle intending to overtake the slow vehicle ahead safely against the traffic in the opposite direction. The overtaking sight distance or passing sight distance is measured along the center line of the road over which a driver with his eye level 1.2 m above the road surface can see the top of an object 1.2 m above the road surface. The factors that affect the OSD are

- Velocities of the overtaking vehicle, overtaken vehicle and of the vehicle coming in the opposite direction.
- Spacing between vehicles, which in-turn depends on the speed
- Skill and reaction time of the driver
- Rate of acceleration of overtaking vehicle
- Gradient of the road



$$OSD = \underbrace{v_b t}_{(d_1)} + \underbrace{2s + v_b\sqrt{\frac{4s}{a}}}_{(d_2)} + \underbrace{vT}_{(d_3)}$$

Where v_b = velocity of the slow moving vehicle in m/sec

t = the reaction time of the driver in (2 sec)

v = velocity of fast moving vehicle in m/sec

s = spacing between the two vehicle in meter = $(0.7v_b + l)$

and a = overtaking vehicles acceleration in m/sec^2.

In case the speed of the overtaken vehicle v_b is not given, it can be assumed that it moves 16 kmph slower the design speed. The acceleration values of the faster vehicle depends on its speed.

The following table shows the relationship between them

Maximum overtaking acceleration at different speeds

Speed (kmph)	Maximum overtaking acceleration (m/sec^2)
25	1.41
30	1.30
40	1.24
50	1.11
65	0.92
80	0.72
100	0.53

Note :

- On divided highways, d_3 need not be considered.
- On divided highways with four or more lanes, IRC suggests that it is not necessary to provide the OSD, but only SSD is sufficient.

1.3.3 Overtaking Zones

Overtaking zones are provided when OSD cannot be provided throughout the length of the highway. These are zones dedicated for overtaking operation, marked with wide roads. The desirable length of overtaking zones is five time OSD and the minimum is three times OSD.

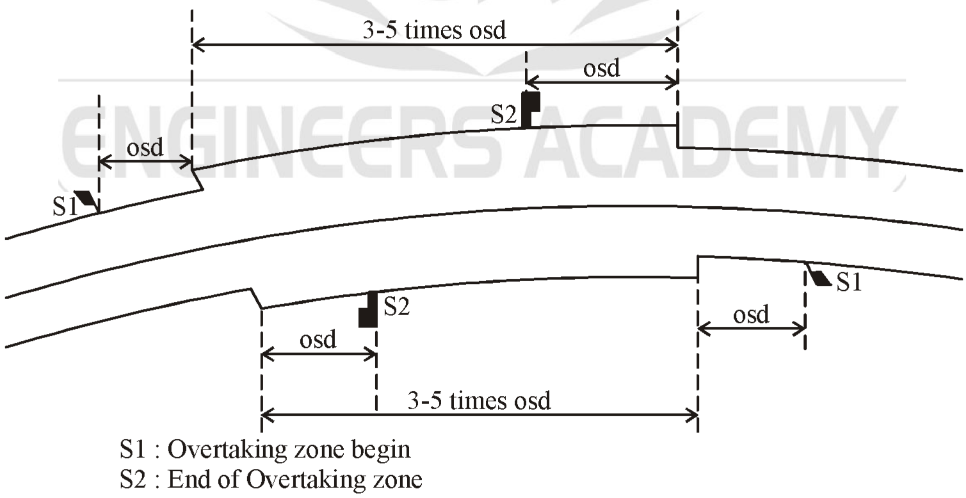


Fig. : Overtaking zones

1.3.4 Sight Distance at Intersections

At intersections where two or more roads meet, visibility should be provided for the drivers approaching the intersection from either sides. They should be able to perceive a hazard and stop the vehicle if required. Stopping sight distance for each road can be computed from the design speed. The sight distance should be provided such that the drivers on either side should be able to see each other :

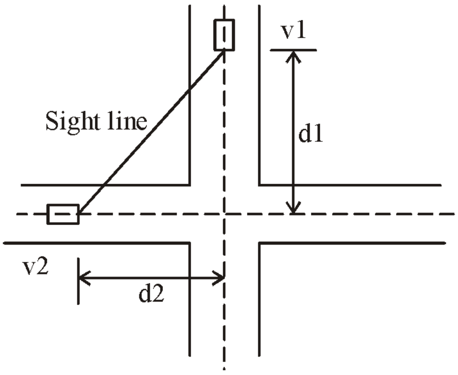


Fig. : Sight distance at intersections

1.4 HORIZONTAL ALIGNMENT

Horizontal alignment is one of the most important features influencing the efficiency and safety of a highway. A poor design will result in lower speeds and resultant reduction in highway performance in terms of safety and comfort.

1.4.1 Design Speed

Indian Road Congress (IRC) has classified the terrains into four categories, namely plain, rolling, mountainous, and steep based on the cross slope as given in table. Based on the type of road and type of terrain the design speed varies. The IRC has suggested desirable or ruling speed as well as minimum suggested design speed and is tabulated in table.

Terrain classification

Terrain classification	Cross slope (%)
Plain	0-10
Rolling	10-25
Mountainous	25-60
Steep	> 60

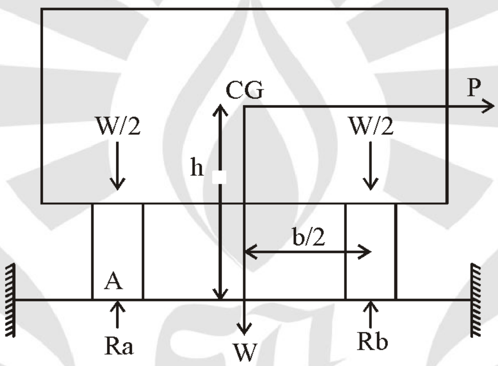
Design speed in km/hr as per IRC (ruling and minimum)

Type	Plain	Rolling	Hilly	Steep
NH & SH	100-80	80-65	50-40	40-30
MDR	80-65	65-50	40-30	30-20
ODR	65-50	50-40	30-25	25-20
VR	50-40	40-35	25-20	25-20

Type	Design speed (kmph)
Arterial roads	80
Sub-arterial roads	60
Collector streets	50
Local streets	30

1.4.2 Horizontal Curve

The presence of horizontal curve imparts centrifugal force which is a reactive force acting outward on a vehicle negotiating it. Centrifugal force depends on speed and radius of the horizontal curve and is counteracted to a certain extent by transverse friction between the tyre and pavement surface. On a curved road, this force tends to cause the vehicle to overturn or to slide outward from the centre of road curvature. For proper design of the curve, an understanding of the forces acting on a vehicle taking a horizontal curve is necessary. Various forces acting on the vehicle is illustrated in the figure. They are the centrifugal force (P) acting outward, weight of the vehicle (W) acting downward, and the reaction of the ground on the wheels (R_A and R_B). The centrifugal force and the weight is assumed to be from the centre of gravity which is at h units above the ground. Let the wheel base be assumed as b units



For safety against overturning the following condition must be satisfied

$$\frac{b}{2h} > \frac{v^2}{gR}$$

For safety against skidding the following condition must be satisfied

$$f > \frac{v^2}{gR}$$

1.4.3 Super-Elevation

Super-elevation or cant or banking is the transverse slope provided at horizontal curve to counteract the centrifugal force, by raising the outer edge of the pavement with respect to the inner edge, throughout the length of the horizontal curve. When the outer edge is raised, a component of the weight will be complimented in counteracting the effect of centrifugal force. In order to find out how much this raising should be, the following analysis may be done. The forces acting on a vehicle while taking a horizontal curve with superelevation is shown in figure

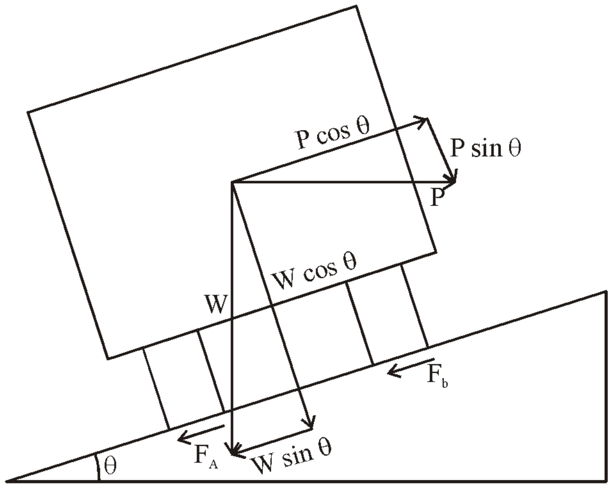


Fig. : Analysis of super-elevation

At equilibrium, by resolving the forces parallel to the surface of the pavement we get

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

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

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

Dividing by W cos θ,

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

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

where, P = centrifugal force acting horizontally out-wards through the center of gravit

W = weight of the vehicle acting down-wards through the center of gravity

F = friction force between the wheels and the pavement, along the surface inward.

1.4.3.1 Design of super-elevation (e)

For fast moving vehicles, providing higher superelevation without considering coefficient of friction is safe, i.e. centrifugal force is fully counteracted by the weight of the vehicle or superelevation. For slow moving vehicles, providing lower superelevation considering coefficient of friction is safe, *i.e.* centrifugal force is counteracted by superelevation and coefficient of friction. IRC suggests following design procedure

Step 1 : Find e for 75 percent of design speed, neglecting f, *i.e.*, $e_1 = \frac{(0.75v)^2}{gR}$

Step 2 : If $e_1 \leq 0.07$, then $e = e_1 = \frac{(0.75v)^2}{gR}$, else if $e_1 > 0.07$ go to step 3.

Step 3 : Find f_1 for the design speed and max e, *i.e.*, $f_1 = \frac{v^2}{gR} - e = \frac{v^2}{gR} - 0.07$. If $f_1 < 0.15$, then the maximum e = 0.07 is safe for the design speed, else go to step 4.

Step 4 : Find the allowable speed v_a for the maximum e = 0.07 and f = 0.15, *i.e.* from equation $v_a = \sqrt{0.22gR}$ If $v_a \geq v$ then the design is adequate, otherwise use speed adopt control measures or look for speed control measures.

1.4.3.2 Maximum and Minimum Superelevation

Depends on

(a) Slow moving vehicle

(b) Heavy loaded trucks with high CG

IRC specifies a maximum super-elevation of 7 percent for plain and rolling terrain, while that of hilly terrain is 10 percent and urban road is 4 percent. The minimum super elevation is 2-4 percent for drainage purpose, especially for large radius of the horizontal curve

1.4.3.3 Attainment of Superelevation

1. **Elimination of the crown of the cambered section by**

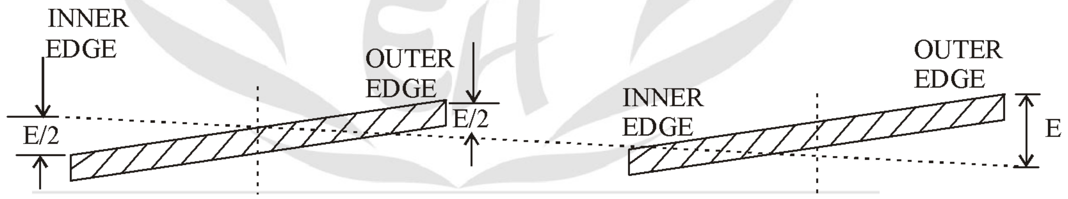
   (a) **Rotating the outer edge about the crown :** The outer half of the cross slope is rotated about the crown at a desired rate such that this surface falls on the same plane as the inner half.

   (b) **Shifting the position of the crown :** This method is also known as diagonal crown method. Here the position of the crown is progressively shifted outwards, thus increasing the width of the inner half of cross section progressively.

2. **Rotation of the pavement cross section to attain full super elevation :** There are two methods of attaining superelevation by rotating the pavement

   (a) **Rotation about the center line :** The pavement is rotated such that the inner edge is depressed and the outer edge is raised both by half the total amount of superelevation, i.e., by E/2 with respect to the centre.

   (b) **Rotation about the inner edge :** Here the pavement is rotated raising the outer edge as well as the centre such that the outer edge is raised by the full amount of superelevation with respect to the inner edge.



1.4.4 Radius of Horizontal Curve

The radius of the horizontal curve is an important design aspect of the geometric design. The maximum comfortable speed on a horizontal curve depends on the radius of the curve. Although it is possible to design the curve with maximum superelevation and coefficient of friction but it is not desirable because re-alignment would be required if the design speed is increased in future

$$R_{ruling} = \frac{v^2_{ruling}}{g(e_{max} + f_{max})}$$

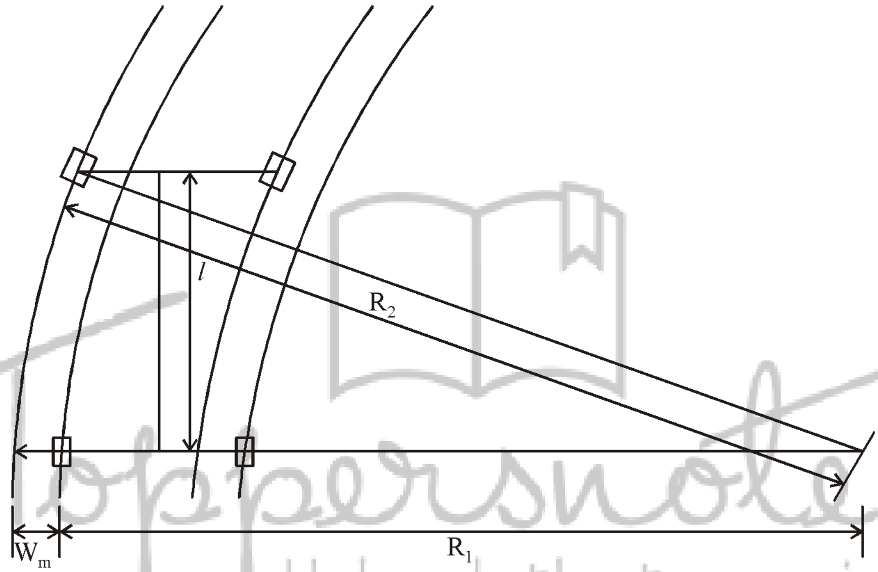
$$R_{minimum} = \frac{v^2_{minimum}}{g(e_{max} + f_{max})}$$

1.4.5 Extra Widening

Extra widening refers to the additional width of carriageway that is required on a curved section of a road over and above that required on a straight alignment. This widening is done due to two reasons: the first and most important is the additional width required for a vehicle taking a horizontal curve and the second is due to the tendency of the drivers to ply away from the edge of the carriageway as they drive on a curve. The first is referred as the mechanical widening and the second is called the psychological widening.

1.4.5.1 Mechanical Widening

When a vehicle negotiates a horizontal curve, the rear wheels follow a path of shorter radius than the front wheels as shown in given figure. This phenomenon is called off tracking, and has the effect of increasing the effective width of a road space required by the vehicle. Therefore, to provide the same clearance between vehicles travelling in opposite direction on curved roads as is provided on straight sections, there must be extra width of carriageway available.



$$W_m = \frac{nl^2}{2R_2}$$

1.4.5.2 Psychological Widening

Widening of pavements has to be done for some psychological reasons also. There is a tendency for the drivers to drive close to the edges of the pavement on curves. Some extra space is to be provided for more clearance for the crossing and overtaking operations on curves. IRC proposed an empirical relation for the psychological widening at horizontal curves W_{ps}

$$W_{ps} = \frac{v}{2.64\sqrt{R}}$$

Therefore, the total widening needed at a horizontal curve W_e is

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

$$= \frac{nl^2}{2R} + \frac{v}{2.64\sqrt{R}}$$

1.4.6 Transition Curves

Transition curve is provided to change the horizontal alignment from straight to circular curve gradually and has a radius which decreases from infinity at the straight end (tangent point) to the desired radius of the circular curve at the other end (curve point) There are five objectives for providing transition curve and are given below

- To introduce gradually the centrifugal force between the tangent point and the beginning of the circular curve, avoiding sudden jerk on the vehicle.This increases the comfort of passengers.
- To enable the driver turn the steering gradually for his own comfort and security,
- To provide gradual introduction of super-elevation and extra widening.
- To enhance the aesthetic appearance of the road.

1.4.6.1 Type of Transition Curve

Different types of transition curves are spiral or clothoid, cubic parabola, and Lemniscate. IRC recommends spiral as the transition curve because

(a) Rate of change of centrifugal acceleration is consistent (smooth)

(b) Radius of the transition curve is infinity at the straight edge and changes to R at the curve point $\left(L_s \propto \frac{1}{R}\right)$ and calculation and field implementation is very easy.

1.4.6.2 Length of Transition Curve

The length of the transition curve should be determined as the maximum of the following three criteria:

(i) rate of change of centrifugal acceleration

(ii) rate of change of superelevation

(iii) an empirical formula given by IRC.

(i) Rate of Change of Centrifugal Acceleration :

At the tangent point, radius is infinity and hence centrifugal acceleration is zero. At the end of the transition, the radius R has minimum value R. The rate of change of centrifugal acceleration should be adopted such that the design should not cause discomfort to the drivers. If c is the rate of change of centrifugal acceleration, it can be written as :

$$c = \frac{\frac{v^2}{R} - 0}{t} = \frac{\frac{v^2}{R} - 0}{\frac{L_s}{v}} = \frac{v^3}{L_s R}$$

- Therefore, the length of the transition curve L_{s_1} in m is

$$L_{s_1} = \frac{v^3}{cR}$$

- Where c is the rate of change of centrifugal acceleration given by an empirical formula suggested by IRC as below

$$c = \frac{80}{75 + 3.6v}$$

$$0.5 \le c \le 0.8$$

where, v = velocity in meter per second

R = radius of curve in meter

L_s = Length of transition curve in meter

(ii) Rate of Introduction of Super-Elevation : Raise (E) of the outer edge with respect to inner edge is given by E = eB = e(W + W_e). the rate of change of this raise from 0 to E is achieved gradually with a gradient of 1 in N over the length of the transition curve (typical range of N is 60 to 150). Therefore, the length of the transition curve L_{s_2} is

$$L_{s_2} = Ne(W + W_e) \qquad \text{(Rotated about inner edge)}$$

or

$$L_{s_2} = \frac{Ne(W + We)}{2} \qquad \text{(Rotated about center line)}$$

(iii) By Empirical Formula : IRC suggest the length of the transition curve is minimum for a plain and rolling terrain

$$L_{s_3} = \frac{35v^2}{R}$$

and for steep and hilly terrain is :

$$L_{s_3} = \frac{12.96v^2}{R}$$

and the shift s as :

$$s = \frac{L_s^2}{24R}$$

The length of the transition curve L_s is the maximum of equations i.e.,

$$L_s = \text{Max} : \left(L_{s_1}, L_{s_2}, L_{s_3}\right)$$

1.4.7 Setback Distance (m)

Setback distance m or the clearance distance is the distance required from the centerline of a horizontal curve to an obstruction on the inner side of the curve to provide adequate sight distance at a horizontal curve. The setback distance depends on

1. Sight distance (SSD, ISD and OSD)
2. Radius of the curve
3. Length of the curve

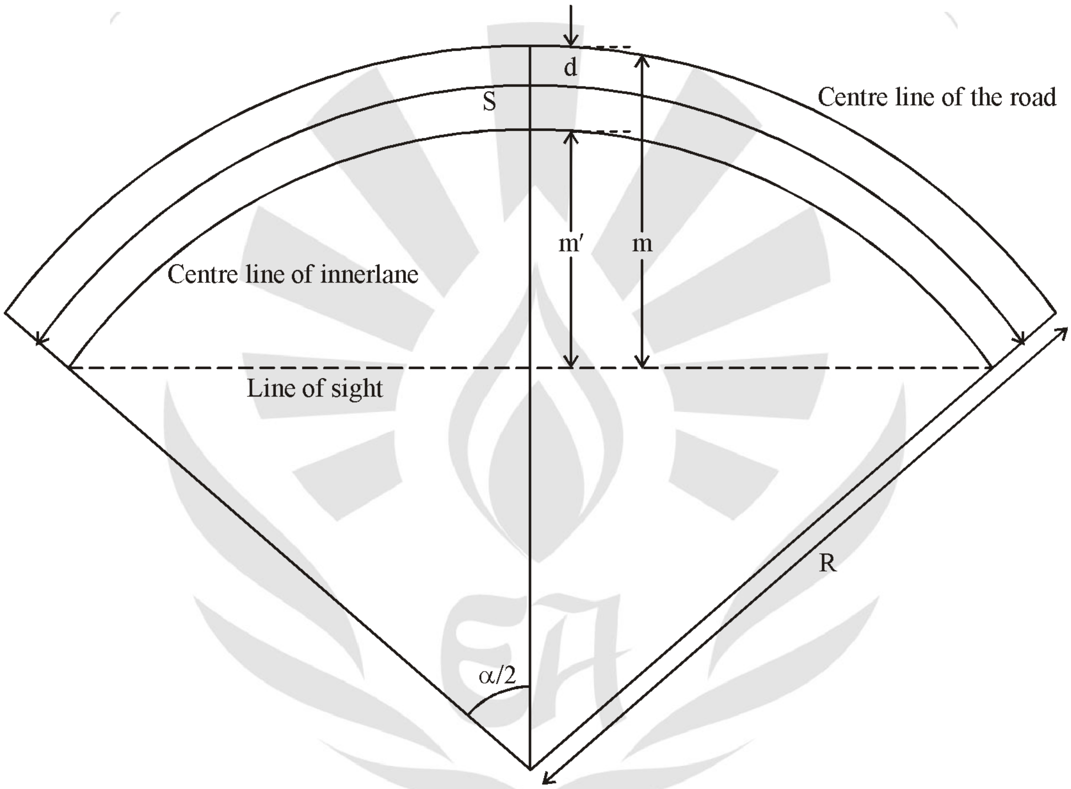
Case (a) $L_s < L_c$

For single lane roads $\alpha = \frac{S}{R}\text{ Radians} = \frac{180S}{\pi R}\text{ Degrees}$

$$\frac{\alpha}{2} = \frac{180S}{2\pi R}\text{ Degrees}$$

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

For multi lane roads, if d is the distance between centerline of the road and the centerline of the inner lane, then



Set-back for multi-lane roads ($L_s < L_c$)

$$m = R-(R-d)\cos\left(\frac{180S}{2\pi(R-d)}\right)$$

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

Case (b) $L_s > L_c$

For single lane $m_1 = R - R\cos(\alpha/2)$

$$m_2 = \frac{(S-L_c)}{2}\sin\left(\frac{\alpha}{2}\right)$$

The set back is the sum of m_1 and m_2 given by

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

Where $$\left(\frac{\alpha}{2}\right) = \frac{180L_c}{2\pi R}$$

For multi-lane road $\frac{\alpha}{2} = \frac{180L_c}{2\pi(R-d)}$ and m is given by

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

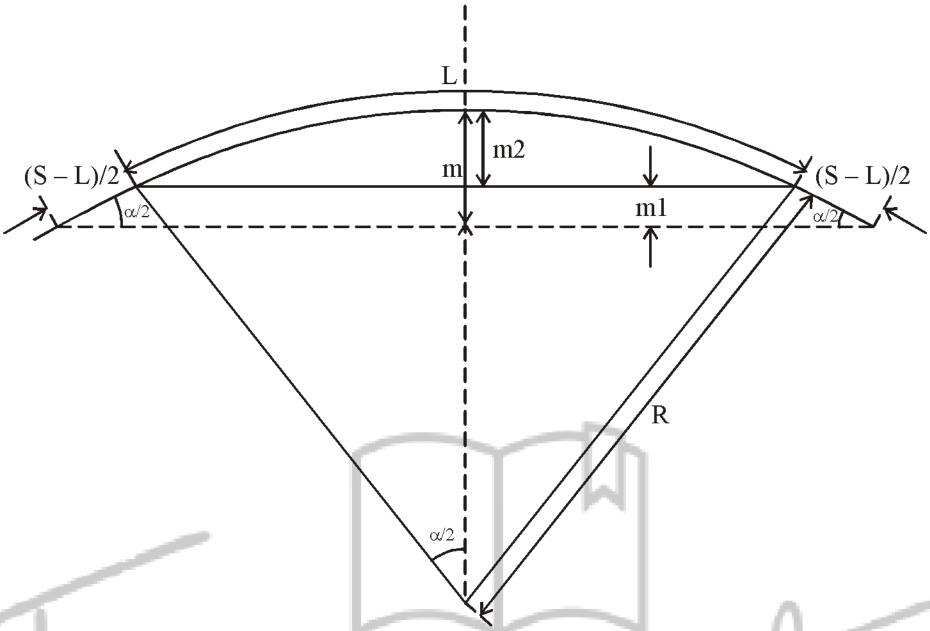
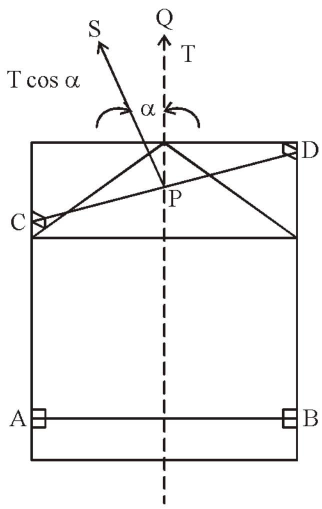


Fig. : Set back for single lane roads ($L_s > L_c$)

1.4.8 Curve Resistance

When the vehicle negotiates a horizontal curve, the direction of rotation of the front and the rear wheels are different. The front wheels are turned to move the vehicle along the curve, whereas the rear wheels seldom turn. This is illustrated in figure. The rear wheels exert a tractive force T in the PQ direction. The tractive force available on the front wheels is Tcos a in the PS direction as shown in the figure. This is less than the actual tractive force, T applied. Hence, the loss of tractive force for a vehicle to negotiate a horizontal curve is

$$CR = T - T\cos\alpha$$



1.5 VERTICAL ALIGNMENT

The vertical alignment of a road consists of gradients(straight lines in a vertical plane) and vertical curves. The vertical alignment is usually drawn as a profile, which is a graph with elevation as vertical axis and the horizontal distance along the centre line of the road as the the horizontal axis. Just as a circular curve is used to connect horizontal straight stretches of road, vertical curves connect two gradients. When these two curves meet, they form either convex or concave. The former is called a summit curve, while the latter is called a valley curve. This section covers a discussion on gradient and vertical curves.

1.5.1 Gradient

Gradient is the rate of rise or fall along the length of the road with respect to the horizontal.

1.5.1.1 Types of Gradient

1. **Ruling gradient**

   The ruling gradient or the design gradient is the maximum gradient with which the designer attempts to design the vertical profile of the road. This depends on the terrain, length of the grade, speed, pulling power of the vehicle and the presence of the horizontal curve.The ruling gradient is adopted by the designer by considering a particular speed as the design speed and for a design vehicle with standard dimensions.

2. **Limiting gradient**

   This gradient is adopted when the ruling gradient results in enormous increase in cost of construction. On rolling terrain and hilly terrain it may be frequently necessary to adopt limiting gradient. But the length of the limiting gradient stretches should be limited and must be sandwiched by either straight roads or easier grades.

3. **Exceptional Gradient**

   Exceptional gradient are very steeper gradients given at unavoidable situations. They should be limited for short stretches not exceeding about 100 metres at a stretch. In mountainous and steep terrain, successive exceptional gradients must be separated by a minimum 100 metre length gentler gradient. At hairpin bends, the gradient is restricted to 2.5%.

   IRC specifications for gradients for different roads

Terrain	Ruling	Limiting	Exceptional
Plain/Rolling	3.3	5.0	6.7
Hilly	5.0	6.0	7.0
Steep	6.0	7.0	8.0

4. **Minimum Gradient :** This is important only at locations where surface drainage is important. Camber will take care of the lateral drainage. But the longitudinal drainage along the side drains require some slope for smooth flow of water. Therefore minimum gradient is provided for drainage purpose and it depends on the rain fall, type of soil and other site conditions.

   According to IRC

   Minimum gradient for concrete drain 1 in 500

   Minimum gradient for earthen drain 1 in 200

1.5.1.2 Grade Compensation

While a vehicle is negotiating a horizontal curve, if there is a gradient also, then there will be increased resistance to traction due to both curve and the gradient. In such cases, the total resistance should not exceed the resistance due to gradient specified. For the design, in some cases this maximum value is limited to the ruling gradient and in some cases as limiting gradient. So if a curve need to be introduced in a portion which has got the maximum permissible gradient, then some compensation should be provided so as to decrease the gradient for overcoming the tractive loss due to curve. Thus grade compensation can be defined as the reduction in gradient at the horizontal curve because of the additional tractive force required due to curve resistance (T -Tcos a), which is intended to offset the extra tractive force involved at the curve.

IRC gave the following specification for the grade compensation

1. Grade compensation is not required for grades flatter than 4% because the loss of tractive force is negligible.
2. Grade compensation is $\frac{30+R}{R}\%$, where R is the radius of the horizontal curve in meters.
3. The maximum grade comprensation is limited to $\frac{75}{R}\%$.

1.5.2 Summit Curve

Summit curves are vertical curves with gradient upwards. They are formed when two gradients meet as illustrated. In figure in any of the following four ways

1. When a positive gradient meets another positive gradient (lesser)
2. When positive gradient meets a flat gradient
3. When an ascending gradient meets a descending gradient
4. When a descending gradient meets another descending gradient. (higher)

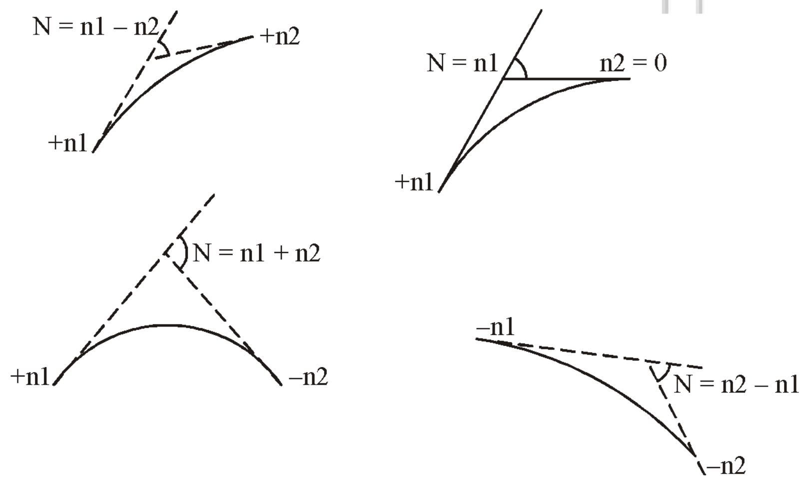
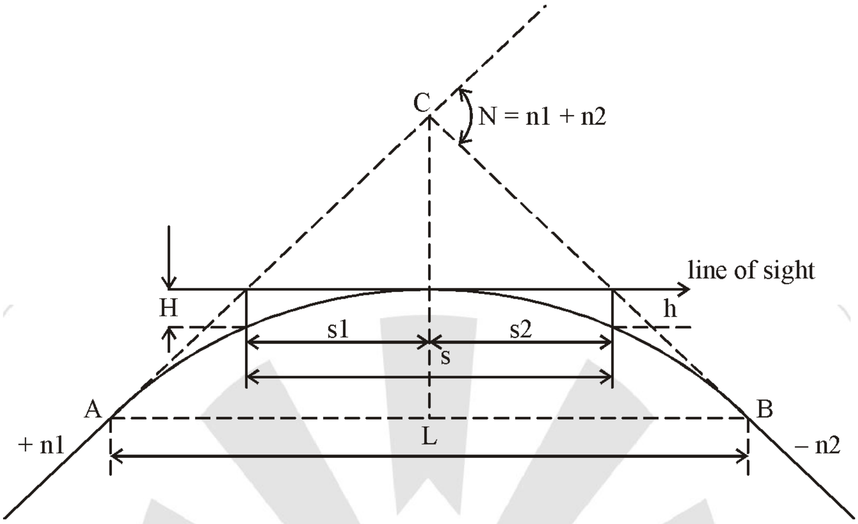


Fig. : Types of summit curves

1.5.2.1 Length of The Summit Curve

Case(a): Length of summit curve is greater than sight distance. The situation when the sight distance is less than the length of the curve is shown in figure



Length of summit curve (L)

$$y = ax^2$$

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

$$h_1 = aS_1^2$$

$$h_2 = aS_2^2$$

$$S_1 = \sqrt{\frac{h_1}{a}}$$

$$S_2 = \sqrt{\frac{h_2}{a}}$$

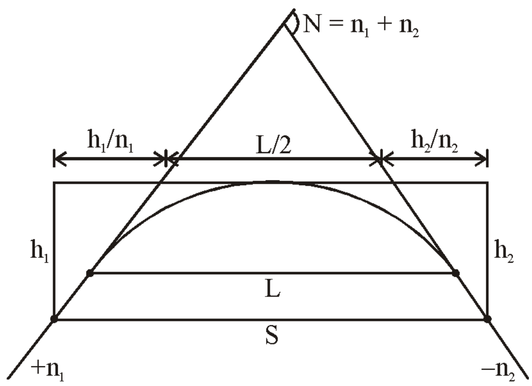
$$S_1 + S_2 = \sqrt{\frac{h_1}{a}} + \sqrt{\frac{h_2}{a}}$$

$$S^2 = \left(\frac{1}{\sqrt{a}}\right)^2 \left(\sqrt{h_1} + \sqrt{h_2}\right)^2$$

$$S^2 = \frac{2L}{N}\left(\sqrt{h_1} + \sqrt{h_2}\right)^2$$

$$L = \frac{NS^2}{2\left(\sqrt{h_1} + \sqrt{h_2}\right)^2}$$

Case(b): Length of summit curve less than sight distance. The second case is illustrated in figure



Length of summit curve (L)

From the basic geometry, one can write

$$S = \frac{L}{2} + \frac{h_1}{n_1} + \frac{h_2}{n_2} = \frac{L}{2} + \frac{h_1}{n_1} + \frac{h_2}{N - n_2}$$

Therefore for a given L, h_1 and h_2 to get minimum S, differentiate the above equation with respect to h_1 and equate it to zero. Therefore,

$$\frac{dS}{dh_1} = \frac{-h_1}{n_1^2} + \frac{h_2}{N - n_1^2} = 0$$

Solving for n_1, $$n_1 = \frac{N\sqrt{h_1 h_2} - h_1 N}{h_2 - h_1}$$

Now we can substitute n back to get the value of minimum value of L for a given n_1, n_2, h_1 and h_2.

Therefore,

$$S = \frac{L}{2} + \frac{h_1}{\frac{N\sqrt{h_1 h_2} - h_1 N}{h_2 - h_1}} + \frac{h_2}{N - \frac{N\sqrt{h_1 h_2} - h_1 N}{h_2 - h_1}}$$

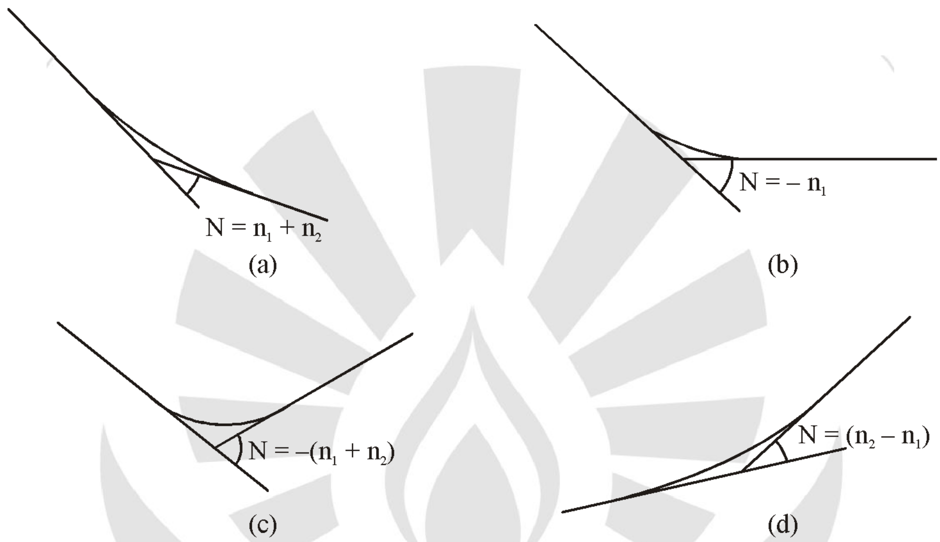
Solving for L, $$L = 2S - \frac{\left(\sqrt{2h_1} + \sqrt{2h_2}\right)^2}{N}$$

When stopping sight distance is considered the height of driver's eye above the road surface (h_1) is taken as 1.2 meters, and height of object above the pavement surface (h_2) is taken as 0.15 meters. If overtaking sight distance is considered, then the value of driver's eye height (h_1) and the height of the obstruction (h_2) are taken equal as 1.2 meters.

1.5.3 Valley Curve

Valley curve or sag curves are vertical curves with convexity downwards. They are formed when two gradients meet as illustrated in figure in any of the following four ways

1. when a descending gradient meets another descending gradient (lesser)
2. when a descending gradient meets a flat gradient
3. when a descending gradient meets an ascending gradient
4. when an ascending gradient meets another ascending (higher)



Types of valley curve

1.5.3.1 Length of The Valley Curve

The valley curve is made full transitional by providing two similar transition curves of equal length. The transitional curve is set out by a cubic parabola $y = bx^3$ where $b = \frac{2N}{3L^2}$. The length of the valley transition curve is designed based on two criteria :

1. **Comfort criteria :** Allowable rate of change of centrifugal acceleration is limited to a comfortable level of about 0.6 m/sec^3. The length of the valley curve based on the rate of change of centrifugal acceleration that will ensure comfort :

   Let c is the rate of change of acceleration, R the minimum radius of the curve, v is the design speed and t is the time, then c is given as

$$c = \frac{\frac{v^2}{R} - 0}{t} = \frac{\frac{v^2}{R} - 0}{\frac{L_s}{v}} = \frac{v^3}{L_s R}$$

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

For a cubic parabola, the value of R for length L_s is given by :

$$R = \frac{L_s}{N}$$

$$L_s = \frac{v^3}{c\frac{L_s}{N}}$$

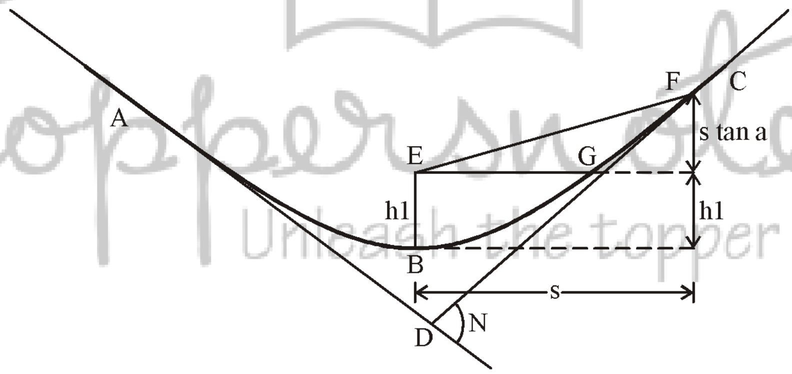
$$L_s = \sqrt{\frac{Nv^3}{c}}$$

Where L is the total length of valley curve, N is the deviation angle in radians or tangent of the deviation angle or the algebric difference in grades, and c is the allowable rate of change of centrifugal acceleration which may be taken as $0.6 m/sec^3$.

2. **Safety criteria :** The driver should have adequate headlight sight distance at any part of the country. Length of the valley curve for headlight distance may be determined for two conditions :

Case 1 : Length of valley curve greater than stopping sight distance (L > S)

The total length of valley curve L is greater than the stopping sight distance SSD. The sight distance available will be minimum when the vehicle is in the lowest point in the valley. This is because the beginning of the curve will have infinite radius and the bottom of the curve will have minimum radius which is a property of the transition curve. From the geometry of the figure, we have :



Valley curve, L > S

$$h_1 + S \tan \alpha = aS^2$$

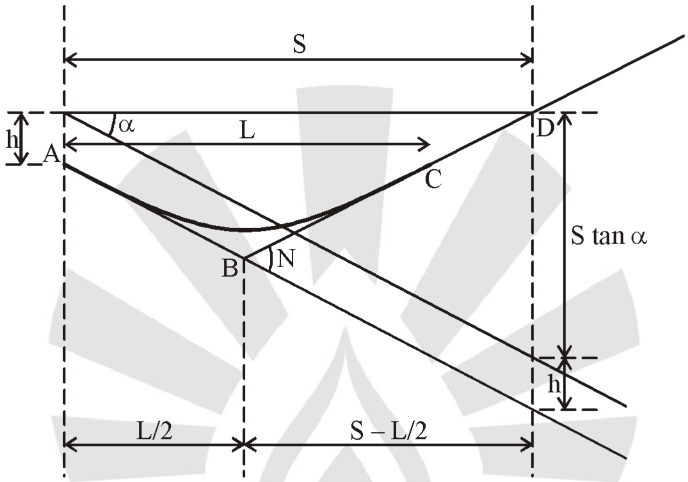
$$= \frac{NS^2}{2L}$$

$$L = \frac{NS^2}{2h_1 + 2S\tan\alpha}$$

Where N is the deviation angle in radians, h_1 is the height of headlight beam, α is the head beam inclination in degrees and S is the sight distance. The inclination α is approx 1 degree.

Case 2 : Length of valley curve less than stopping sight distance (L < S)

The length of the curve L is less than SSD. In this case the minimum sight distance is from the beginning of the curve. The important points are the beginning of the curve and the bottom most part of the curve. If the vehicle is at the bottom of the curve, then its headlight beam will reach far beyond the endpoint of the curve whereas, if the vehicle is at the beginning of the curve, then the headlight beam will hit just outside the curve. Therefore, the length of the curve is derived by assuming the vehicle at the beginning of the curve. The case is shown in below figure. From the figure,



Valley curve (L < S)

$$h_1 + s \tan \alpha = \left(S - \frac{L}{2}\right)N$$

$$L = 2S - \frac{2h_1 + 2S\tan\alpha}{N}$$

1.6 PAVEMENT DESIGN

1.6.1 Requirements of a Pavement

The pavement should meet the following requirements :

- Sufficient thickness to distribute the wheel load stresses to a safe value on the sub-grade soil.
- Structurally strong to withstand all types of stresses imposed upon it
- Adequate coefficient of friction to prevent skidding of vehicles.
- Smooth surface to provide comfort to road users even at high speed.
- Produce least noise from moving vehicles.
- Dust proof surface so that traffic safety is not impaired by reducing visibility.
- Impervious surface, so that sub-grade soil is well protected
- Long design life with low maintenance cost

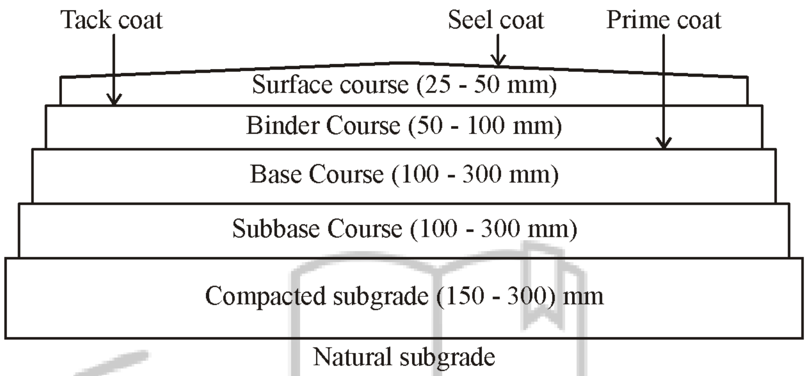
1.6.2 Types of Pavements

The pavements can be classified based on the structural performance into two, flexible pavements and rigid pavements.

1.6.2.1 Flexible Pavements

Flexible pavements are those, which on the whole have low or negligible flexural strength and are rather flexible in their structural action under the loads. The flexible pavement layers may reflect the non-recoverable as well as coverable deformations of the lower layers including the subgrade on to the upper layers and also to the pavement surface.

In flexible pavements, wheel loads are transferred by grain-to-grain contact of the aggregate through the granular structure. The flexible pavement, having less flexural strength, acts like a flexible sheet (e.g. bituminous road).



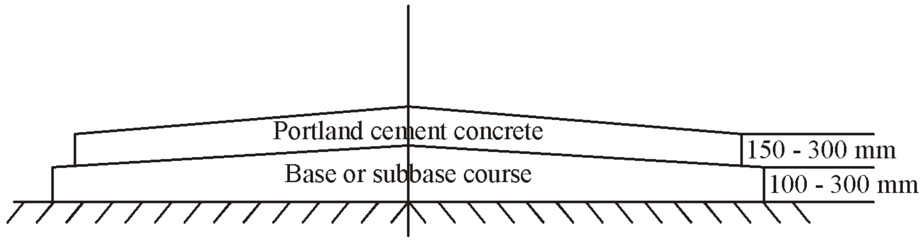
Typical cross section of a flexible pavement

1.6.2.2 Rigid Pavements

Rigid pavements are those which possess noteworthy flexural strength or flexural rigidity. The rigid pavements aregenerally made of Portland cement concrete (CC) and are therefore called 'CC pavements'.

In rigid pavements, wheel loads are transferred to sub-grade soil by flexural strength of the pavement and the pavement acts like a rigid plate (e.g. cement concrete roads).

Other types of pavement structure include (a) 'semi-rigid pavement' or 'composite pavement' and (b) Interlocking cement Concrete Block Pavement (ICBP). However these types of pavements are less common when compared to flexible and rigid pavements.



Typical cross section of rigid pavement

Seal Coat : Seal coat is a thin surface treatment used to water-proof the surface and to provide skid resistance.

Tack Coat : Tack coat is a very light application of asphalt, usually asphalt emulsion diluted with water. It provides proper bonding between two layer of binder course and must be thin, uniformly cover the entire surface, and set very fast.

Prime Coat : Prime coat is an application of low viscous cutback bitumen to an absorbent surface like granular bases on which binder layer is to be placed. It provides bonding between two layers. Unlike tack coat, prime coat penetrates into the layer below, plugs the voids, and forms a water tight surface.

Surface Course : Surface course is the layer directly in contact with traffic loads and generally contains superior quality materials. They are usually constructed with dense graded asphalt concrete(AC). The functions and requirements of this layer are :

- It provides characteristics such as friction, smoothness, drainage, etc. Also it will prevent the entrance of excessive quantities of surface water into the underlying base, sub-base and sub-grade.
- It must be tough to resist the distortion under traffic and provide a smooth and skid- resistant riding surface.
- It must be water proof to protect the entire base and sub-grade from the weakening effect of water.

Binder Course : This layer provides the bulk of the asphalt concrete structure. It's chief purpose is to distribute load to the base course. The binder course generally consists of aggregates having less asphalt and doesn't require quality as high as the surface course, so replacing a part of the surface course by the binder course results in more economical design.

Base Course : The base course is the layer of material immediately beneath the surface of binder course and it provides additional load distribution and contributes to the sub-surface drainage It may be composed of crushed stone, crushed slag, and other untreated or stabilized materials.

Sub-Base Course : The sub-base course is the layer of material beneath the base course and the primary functions are to provide structural support, improve drainage and reduce the intrusion of fines from the sub-grade in the pavement structure If the base course is open graded, then the sub-base course with more fines can serve as a filler between sub-grade and the base course A sub-base course is not always needed or used. For example, a pavement constructed over a high quality, stiff sub-grade may not need the additional features offered by a sub-base course. In such situations, sub-base course may not be provided

Sub-grade : The top soil or sub-grade is a layer of natural soil prepared to receive the stresses from the layers above. It is essential that at no time soil sub-grade is overstressed. It should be compacted to the desirable density, near the optimum moisture content.

1.6.3 California Bearing Ratio Test

California Bearing Ratio (CBR) test was developed by the California Division of Highway as a method of classifying and evaluating soil-sub grade and base course materials for flexible pavements. CBR test, an empirical test, has been used to determine the material properties for pavement design. Empirical tests measure the strength of the material and are not a true representation of the resilient modulus. It is a penetration test wherein a standard piston, have an area of 3 in^2 (or 50 mm diameter), is used to penetrate the soil at a standard rate of 1.25 mm/minute. The pressure up to a penetration of 2.5 mm and it's ratio to the bearing value of a standard crushed rock is termed as the CBR.

$$\text{CBR} = \frac{\text{load carries by specimen}}{\text{load carries by standard specimen}} \times 100$$

CBR value is expressed as a percentage of the actual load causing the penetrations of 2.5 mm or 5.0 mm to the standard loads. Two values of CBR will be obtained. If the value of 2.5 mm is greater than that of 5.0 mm penetration, the former is adopted. If the CBR value obtained from test at 5.0 mm penetration is higher than that at 2.5 mm, then the test is to be repeated for checking. If the check test again gives similar results, then higher value obtained at 5.0 mm penetration is reported as the CBR value. The average CBR value of three test specimens is reported as the CBR value of the sample.

1.6.4 Plate Bearing Test

Plate bearing test is used to evaluate the support capability of sub-grades, bases and in some cases, complete pavement. Data from the tests are applicable for the design of both flexible and rigid pavements. In plate bearing test, a compressive stress is applied to the soil or pavement layer through rigid plates relatively large size and the deflections are measured for various stress values. The deflection level is generally limited to a low value, in the order of 1.25 to 5 mm and so the deformation caused may be partly elastic and partly plastic due to compaction of the stressed mass with negligible plastic deformation. The plate-bearing test has been devised to evaluate the supporting power of sub grades or any other pavement layer by using plates of larger diameter. The plate-bearing test was originally meant to find the modulus of sub grade reaction in the Westergaard's analysis for wheel load stresses in cement concrete pavements.

A graph is plotted with the mean settlement versus bearing pressure (load per unit area). The pressure corresponding to a settlement is obtained from this graph. The modulus of subgrade reaction is calculated from the relation.

$$K = \frac{P}{0.125} \text{ kg/cm}^2\text{/cm}$$

1.6.5 Aggregate Tests

1. **Crushing Test :** One of the modes in which pavement material can fail is by crushing under compressive stress. A test is standardized by IS : 2386 part-IV and used to determine the crushing strength of coarse aggregates. The aggregate crushing value provides a relative measure of resistance to crushing under gradually applied crushing load. The test consists of subjecting the specimen of aggregate in standard mould to a compression test under standard load conditions. Dry aggregates passing through 12.5 mm sieves and retained 10 mm sieves are filled in a cylindrical measure of 11.5 mm diameter and 18 cm height in three layers. Each layer is tampered 25 times with at standard tamping rod. The test sample is weighed and placed in the test cylinder in three layers each layer being tampered again. The specimen is subjected to a compressive load of 40 tonnes gradually applied by plunger at the rate of 4 tonnes per minute. Then crushed aggregates are then sieved through 2.36 mm sieve and weight of passing material (W_2) is expressed as percentage of the weight of the total sample (W_1) which is the aggregate crushing value.

   $$\text{Aggregate crushing value} = \frac{W_2}{W_1} \times 100$$

   A value less than 10% signifies an exceptionally strong aggregate while above 35% would normally be regarded as weak aggregates.

2. **Abrasion test :** Abrasion test is carried out to test the hardness property of aggregates and to decide whether they are suitable for different pavement construction works. Los Angeles abrasion test is a preferred one for carrying out the hardness property and has been standardised in India (IS:2386 part-IV). The principle of Los Angeles abrasion test is to find the percentage wear due to relative rubbing action between the aggregate and steel balls used as abrasive charge.

   A maximum value of 50 percent is allowed for WBM base course in Indian conditions. For bituminous concrete, a maximum value of 30% is specified.

3. **Impact Test :** The aggregate impact test is carried out to evaluate the resistance to impact of aggregates. Aggregates passing 12.5 mm sieve and retained on 10 mm sieve is filled in a cylindrical steel cup of internal dia 10.2 mm and depth 5 cm which is attached to a metal base of impact testing machine. The material is filled in 3 layers where each layer is tamped for 25 number of blows. Metal hammer of weight 13.5 to 14 kg is arranged to drop with a free fall of 38.0 cm by vertical guides and the test specimen is subjected to 15 number of blows. The crushed aggregate is allowed to pass through 2.36 mm IS sieve and the impact value is measured as percentage of aggregates passing sieve (W_2) to the total weight of the sample (W_1).

$$\text{Aggregate impact value} = \frac{W_2}{W_1} \times 100$$

Aggregates to be used for wearing course, the impact value shouldn't exceed 30 percent. For bituminous macadam the maximum permissible value is 35 percent. For Water bound macadam base courses the maximum permissible value defined by IRC is 40 percent.

4. **Soundness test :** Soundness test is intended to study the resistance of aggregates to weathering action, by conducting accelerated weathering test cycles. The Porous aggregates subjected to freezing and thawing are likely to disintegrate prematurely. To ascertain the durability of such aggregates, they are subjected to an accelerated soundness test as specified in IS : 2386 part-V. Aggregates of specified size are subjected to cycles of alternate wetting in a saturated solution of either sodium sulphate or magnesium sulphate for 16 – 18 hours and then dried in oven at 105 – 110ºC to a constant weight. After five cycles, the loss in weight of aggregates is determined by sieving out all undersized particles and weighing and the loss in weight should not exceed 12 percent when tested with sodium sulphate and 18 percent with magnesium sulphate solution.

5. **Shape Tests :** The particle shape of the aggregate mass is determined by the percentage of flaky and elongated particles in it. Aggregates which are flaky or elongated are detrimental to higher workability and stability of mixes. The flakiness index is defined as the percentage by weight of aggregate particles whose least dimension is less than 0.6 times their mean size. Test procedure had been standardised in India (IS:2386 part-I). The elongation index of an aggregate is defined as the percentage by weight of particles whose greatest dimension (length) is more than 1.8 times their mean size. This test is applicable to aggregates larger than 6.3 mm. This test is also specified in (IS:2386 Part-I). However there are no recognised limits for the elongation index.

6. **Specific Gravity and water absorption :** The specific gravity and water absorption of aggregates are important properties that are required for the design of concrete and bituminous mixes. The specific gravity of a solid is the ratio of its mass to that of an equal volume of distilled water at a specified temperature. Because the aggregates may contain water-permeable voids, so two measures of specific gravity of aggregates are used: apparent specific gravity and bulk specific gravity. Apparent Specific Gravity, G_{app}, is computed on the basis of the net volume of aggregates i.e., volume excluding water-permeable voids. Thus

$$G_{app} = \frac{M_D / V_N}{\rho}$$

Where M_D = dry mass of the aggregate

V_N = net volume of the aggregates excluding the volume of the absorbed matter

r = mass density of water.

Bulk Specific Gravity, G_{bulk}, is computed on the basis of the total volume of aggregates including water permeable voids. Thus

$$G_{bulk} = \frac{M_D / V_B}{\rho}$$

Where V_B = total volume of the aggregates including the volume of absorbed water.

Water absorption : The difference between the apparent and bulk specific gravities is nothing but the water permeable voids of the aggregates. We can measure the volume of such voids by weighing the aggregates dry and in a saturated, surface dry condition, with all permeable voids filled with water. The difference of the above two is M_W. M_W is the weight of dry aggregates minus weight of aggregates saturated surface dry condition. Thus

$$\text{Water absorption} = \frac{M_W}{M_D} \times 100$$

The specific gravity of aggregates normally used in road construction ranges from about 2.5 to 2.9. Water absorption values ranges from 0.1 to about 2.0 percent for aggregates normally used in road surfacing.

7. **Angularity number :** The apparatus for testing the angularity number consists of a metal cylinder of capacity 3 litre, tamping rod and a metal scoop. The test sample is sieved and a specified size ranges of the aggregate, such as 16 - 20 mm, 12.5 - 16 mm, etc. are used for the test. A scoop full of this single size aggregate is placed in the cylinder and tamped 100 times by the rod. Second and third layers are placed in the cylinder and tamped 100 times by the rod. Second and third layers are placed and tamped similarly and the excess aggregate is struck off level to the top surface of the cylinder. The weight of aggregate in the cylinder is found to be W g. Then the cylinder is emptied and the weight of water filling the cylinder is determined = C g. The specific gravity G_a of the aggregate is also determined.

   The angularity number, AN is found from the formula :

$$AN = 67 - \frac{100W}{CG_a}$$

1.7 BITUMEN

1.7.1 Different forms of Bitumen

1. **Cutback bitumen :** Normal practice is to heat bitumen to reduce its viscosity. In some situations preference is given to use liquid binders such as cutback bitumen. In cutback bitumen suitable solvent is used to lower the viscosity of the bitumen. From the environmental point of view also cutback bitumen is preferred. The solvent from the bituminous material will evaporate and the bitumen will bind the aggregate.

   Cutback bitumen is used for cold weather bituminous road construction and maintenance. The distillates used for preparation of cutback bitumen are naphtha, kerosene, diesel oil, and furnance oil. There are different types of cutback bitumen like rapid curing (RC), medium curing (MC), and slow curing (SC). IRC is recommended RC for surface dressing and patchwork. MC is recommended for premix with less quantity of fine aggregates. SC is used for premix with appreciable quantity of fine aggregates.

2. **Bitumen Emulsion :** Bitumen emulsion is a liquid product in which bitumen is suspended in a finely divided condition in an aqueous medium and stabilised by suitable material. Normally cationic type emulsions are used in India. The bitumen content in the emulsion is around 60% and the remaining is water. When the emulsion is applied on the road it breaks down resulting in release of water and the mix starts to set. The time of setting depends upon the grade of bitumen. The viscosity of bituminous emulsions can be measured as per IS: 8887-2004.

   Three types of bituminous emulsions are available, which are Rapid setting (RS), Medium setting (MS), and Slow setting (SC). Bitumen emulsions are ideal binders for hill road construction. Where heating of bitumen or aggregates are difficult. Rapid setting emulsions are used for surface dressing work. Medium setting emulsions are preferred for premix jobs and patch repairs work. Slow setting emulsions are preferred in rainy season.

1.7.2 Tests on Bitumen

1. **Penetration Test :** It measures the hardness or softness of bitumen by measuring the depth in tenths of a millimeter to which a standard loaded needle will penetrate vertically in 5 seconds. BIS had standardised the equipment and test procedure. The penetrometer consists of a needle assembly with a total weight of 100 gm and a device for releasing and locking in any position. The bitumen is softened to a pouring consistency, stirred throughly and poured into containers at a depth at least 15 mm in excess of the expected penetration. The test should be conducted at a specified temperature of 25ºC. It may be noted that penetration value is largely influenced by any inaccuracy with regards to pouring temperature, size of the needle, weight placed on the needle and the test temperature. A grade of 40/50 bitumen means the penetration value is in the range 4mm to 5mm at standard test conditions. In hot climates, a lower penetration grade is preferred.
2. **Ductility Test :** Ductility is the property of bitumen that permits it to undergo great deformation or elongation. Ductility is defined as the distance in cm, to which a standard sample or briquette of the material will be elongated without breaking. Dimension of the briquette thus formed is exactly 1 cm square. The bitumen sample is heated and poured in the mould assembly placed on a plate. These samples with moulds are cooled in the air and then in water bath at 27ºC temperature. The excess bitumen is cut and the surface is leveled using at hot knife. Then the mould with assembly containing sample is kept in water bath of the ductility machine for about 90 minutes. The sides of the moulds are removed, the clips are hooked on the machine and the machine is operated. The distance up to the point of breaking of thread is the ductility value which is reported in cm. The ductility value gets affected by factors such as pouring temperature, test temperature, rate of pulling etc. A minimum ductility value of 75 cm has been specified by the BIS.
3. **Softening Point Test :** Softening point denotes the temperature at which the bitumen attains a particular degree of softening under the specifications of test. The test is conducted by using Ring and ball apparatus. A brass ring containing test sample of bitumen is suspended in liquid like water or glycerin at a given temperature. A still ball is placed upon the bitumen sample and the liquid medium is heated at a rate of 5°C per minute. Temperature is noted when the softened bitumen touches the metal plate which is at a specified distance below. Generally, higher softening point indicates lower temperature susceptibility and is preferred in hot climates.
4. **Specific Gravity Test :** In paving jobs, to classify a binder, density property is of great use. In most cases bitumen is weighed, but when used with aggregates, the bitumen is converted to volume using density values. The density of bitumen is greatly influenced by its chemical composition. Increase in aromatic type mineral impurities cause an increase in specific gravity. The specific gravity of

bitumen is defined as the ratio of mass of given volume of bitumen of known content to the mass of equal volume of water at 27°C. The specific gravity can be measured using either pycnometer or preparing a cube specimen of bitumen in semi solid or solid state. The specific gravity of bitumen varies from 0.97 to 1.02.

5. **Viscosity Test :** Viscosity denotes the fluid property of bituminous material and it is a measure of resistance to flow. At the application temperature, this characteristic greatly influences the strength of resulting paving mixes. Low or high viscosity during compaction or mixing has been observed to result in lower stability values. At high viscosity instead of providing a uniform film over aggregates, it will lubricate the aggregate particles.Orifice type viscometers are used to indirectly find the viscosity of liquid binders like cutbacks and emulsions. The viscosity expressed in seconds is the time taken by the 50 ml bitumen material to pass through the orifice of a cup under standard test conditions and specified temperature. Viscosity of a cutback can be measured with either 4.0 mm orifice at 25°C or 10 mm orifice at 25 or 40°C.

6. **Flash and Fire Point Test :** At high temperatures depending upon the grades of bitumen materials leave out volatiles and these volatiles catches fire which is very hazardous and therefore it is essential to qualify the temperature for each bitumn grade. BIS defined the flash point as the temperature at which the vapour of bitumen momentarily catches fire in the form of flash under specified test conditions. The fire point is defined as the lowest temperature under specified test conditions at which the bituminous material gets ignited and burns.

7. **Float Test :** Normally the consistency of bituminous material can be measured either by penetration test or viscosity test. But for certain range of consistencies, these test are not applicable and Float test is used. The apparatus consists of an aluminium float and a brass collar filled with bitumen to be tested. The specimen in the mould is cooled to a temperature of 5°C and screwed in to float. The total test assembly is floated in the water bath at 50°C and the time required for water to pass its way through the specimen blug is noted in seconds and is expressed as the float value.

8. **Water Content Test :** It is desirable that the bitumen contains minimum water content to prevent foaming of the bitumen when it is heated above the boiling point of water. The water in a bitumen is determined by mixing known weight of specimen in a pure petroleum distillate free from water, heating and distilling of the water. The weight of the water condensed and collected is expressed as percentage by weight of the original sample. The allowable maximum water content should not be more than 0.2% by weight.

9. **Loss on Heating Test :** When the bitumen is heated it loses the volatility and gets hardened. About 50 gm of the sample is weighed and heated to a temperature of 163°C for 5 hours in a specified oven designed for this test. The sample specimen is weighed again after the heating period and loss in weight is expressed as percentage by weight of the original sample. Bitumen used in pavement mixes should not indicate more than 1% loss in weight, but for bitumen having penetration values 150 - 200 up to 2% loss in weight is allowed.

10. **Spot test :** This is a test for detecting over heated or ‘cracked’ bitumen. This test is considered to be more sensitive than the solubility test for detection of cracking. About 2.0 g of bitumen is dissolved in 10 ml of naphtha. A drop of this soloution is taken out and placed on a filter paper, the first drop after one hour and second one after 24 hours after the solution is prepared. If the stain of the spot on the filter paper is uniform in colour, the bitumen is accepted as not cracked. But if the spots from dark brown or black circle in the centre with an annular ring of lighter colour surrounding it, the bitumen is considered to be over heated or cracked.

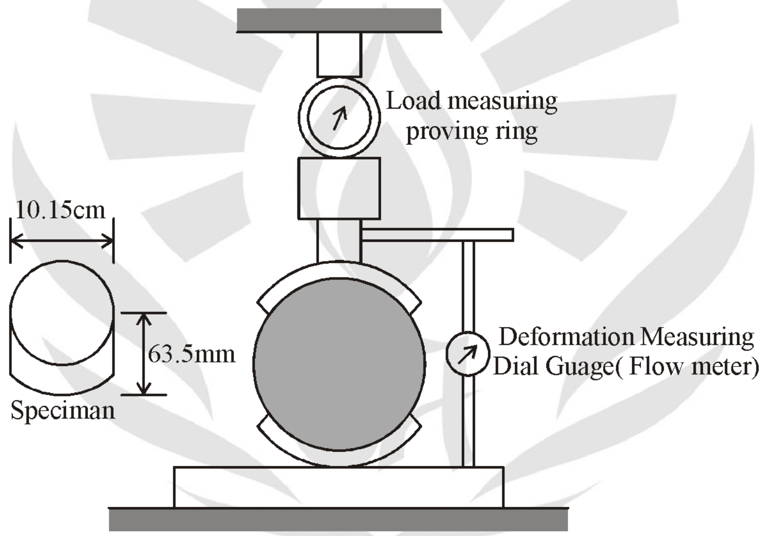
1.7.3 Marshall Mix Design

The mix design (wetmix) determines the optimum bitumen content. This is proceeded by the dry mix design. There are many methods available for mix design which vary in the size of the test specimen, compaction, and other test specifications. Marshall method of mix design is the most popular one and is discussed below.

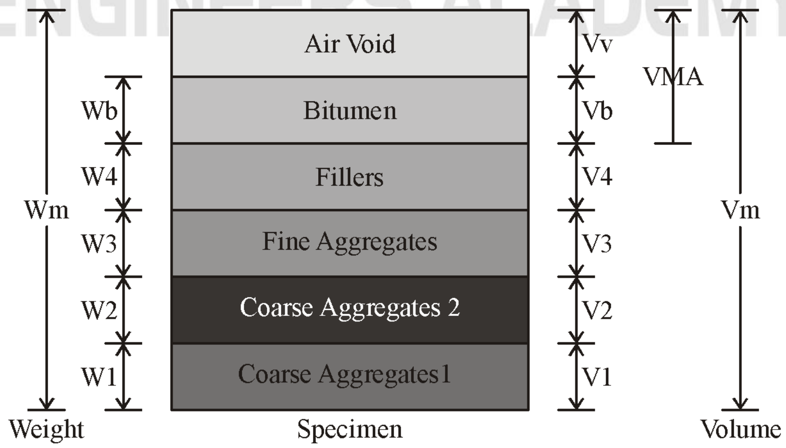
The Marshall stability and flow test provides the performance prediction for the Marshall mix design method. The stability portion of the test measures the maximum load supported by the test specimen at a loading rate of 50.8 mm/minute. Load is applied to the specimen till failure, and the maximum load is designated as stability. During the loading, an attached dial gauge measures the specimen's plastic flow (deformation) as a result of the loading. The flow value is recorded in 0.25 mm (0.01 inch) increments at the same time when the maximum load is recorded.

1.7.3.1 Determine The Properties of The Mix

The properties that are of interest include the theoretical specific gravity G_t, the bulk specific gravity of the mix G_m, percent air voids V_v, percent volume of bitumen V_b, percent void in mixed aggregate VMA and percent voids filled with bitumen VFB. These calculations are discussed next. To understand these calculation a phase diagram is given in Figure



Marshall test set-up



Phase diagram for bituminous mix

Theoretical specific gravity of the mix G_t : Theoretical specific gravity G_t is the specific gravity without considering air voids, and is given by

$$G_t = \frac{W_1 + W_2 + W_3 + W_b}{\frac{W_1}{G_1} + \frac{W_2}{G_2} + \frac{W_3}{G_3} + \frac{W_b}{G_b}}$$

where, W_1 = weight of coarse aggregate in the total mix

W_2 = weight of fine aggregate in the total mix

W_3 = weight of filler in the total mix

W_b = weight of bitumen in the total mix

G_i = apparent specific gravity of C.A., F.A., filler (i = 1, 2, 3)

G_b = apparent specific gravity of bitumen.

Bulk Specific Gravity of mix G_m : The bulk specific gravity or the actual specific gravity of the mix G_m is the specific gravity considering air voids and is found out by :

$$G_m = \frac{W_m}{W_m - W_w}$$

where W_m = weight of mix in air

W_w = weight of mix in water

AIR Voids Percent V_v : Air voids V_v is the percent of air voids by volume in the specimen and is given by

$$V_v = \frac{(G_t - G_m)100}{G_t}$$

where G_t = theoretical specific gravity of the mix

G_m = bulk or actual specific gravity of the mix

Percent Volume of Bitumen V_b : The volume of bitumen V_b is the percent of volume of bitumen to the total volume and given by :

$$V_b = \frac{\frac{W_b}{G_b}}{\frac{W_1 + W_2 + W_3 + W_b}{G_m}}$$

where W_1 = weight of coarse aggregate in the total mix

W_2 = weight of fine aggregate in the total mix

W_3 = weight of filler in the total mix

W_b = weight of bitumen in the total mix

G_b = apparent specific gravity of bitumen

G_m = bulk specific gravity of mix

Voids in Mineral Aggregate VMA : Voids in mineral aggregate VMA is the volume of voids in the aggregates, and is the sum of air voids and volume of bitumen, and is calculated from

$$VMA = V_v + V_b$$

where V_v = percent air voids in the mix

V_b = percent bitumen content in the mix

Voids Filled With Bitumen VFB : Voids filled with bitumen VFB is the voids in the mineral aggregate frame work filled with the bitumen, and is calculated as :

$$VFB = \frac{V_b \times 100}{VMA}$$

where V_b = percent bitumen content in the mix

VMA = percent voids in the mineral aggregate

1.7.3.2 Determine Optimum Bitumen Content

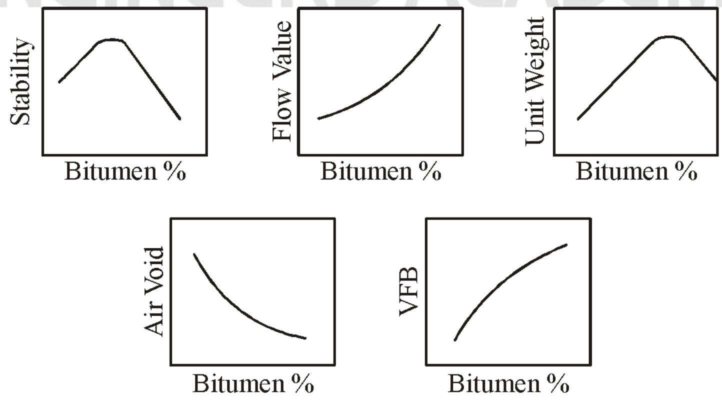
Determine the optimum binder content for the mix design by taking average value of the following three bitumen contents found form the graphs obtained in the previous step.

1. Binder content corresponding to maximum stability.
2. Binder content corresponding to maximum bulk specific gravity (G_m)
3. Binder content corresponding to the median of designed limits of percent air voids (V_v) in the total mix (i.e., 4%)

The stability value, flow value, and VFB are checked with Marshall mix design specification chart given in Table below. Mixes with very high stability value and low flow value are not desirable as the pavements constructed with such mixes are likely to develop cracks due to heavy moving loads.

Marshall mix design specification

Test Property	Specified Value
Marshall stability, kg	340 (minimum)
Flow value, 0.25 mm units	8 – 17
Percent air voids in the mix V_v%	3 – 5
Voids filled with bitumen VFB%	75 – 85



Marshal graphical plots

1.8 FLEXIBLE PAVEMENT DESIGN

1.8.1 Equivalent Single Wheel Load

To carry maximum load with in the specified limit and to carry greater load, dual wheel, or dual tandem assembly is often used. Equivalent single wheel Load (ESWL) is the single wheel load having the same contact pressure, which produces same value of maximum stress, deflection, tensile stress or contact pressure at the desired depth. The procedure of finding the ESWL for equal stress criteria is provided below. This is a semi-rational method, known as Boyd and Foster method, based on the following assumptions :

- Equivalency concept is based on equal stress
- Contact area is circular
- Influence angle is 45°
- Soil medium is elastic, homogeneous, and isotropic

The ESWL is given by :

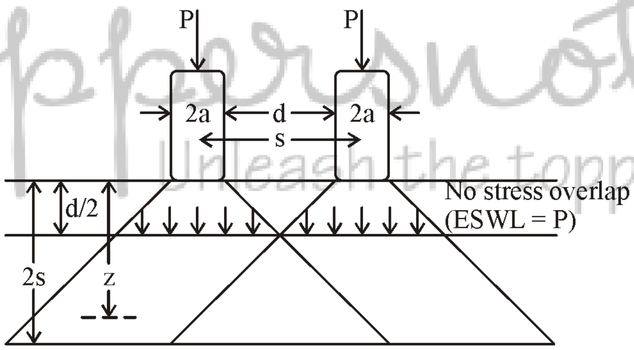
$$\log_{10} \text{ESWL} = \log_{10} P + \frac{0.301 \log_{10}\left(\dfrac{z}{d/2}\right)}{\log_{10}\left(\dfrac{2S}{d/2}\right)}$$

where P = wheel load

S = center to center distance between the two wheels

d = clear distance between two wheels

z = desired depth



ESWL-Equal stress concept

1.8.2 IRC Method of Design of Flexible Pavements

The method considers traffic in terms of the cumulative number of standard axles (8160 kg) to be carried by the pavement during the design life. This requires the following information

1. Initial traffic in terms of (CVPD)
2. Traffic growth rate during the design life
3. Design life in number of years
4. Vehicle damage factor (VDF)
5. Distribution of commercial traffic over the carriage way.

1.9 RIGID PAVEMENT DESIGN

1.9.1 Modulus of Sub-grade Reaction

Westergaard considered the rigid pavement slab as a thin elastic plate resting on soil sub-grade, which is assumed as a dense liquid. The upward reaction is assumed to be proportional to the deflection. Base on this assumption, Westergaard defined a modulus of sub-grade reaction K in kg/cm^3 given by $K = \frac{p}{\Delta}$ where Δ is the displacement level taken as 0.125 cm and p is the pressure sustained by the rigid plate of 75 cm diameter at a deflection of 0.125 cm.

1.9.2 Relative Stiffness of Slab to Sub-grade

A certain degree of resistance to slab deflection is offered by the sub-grade. The sub-grade deformation is same as the slab deflection. Hence the slab deflection is direct measurement of the magnitude of the sub-grade pressure. This pressure deformation characteristic of rigid pavement lead Westergaard to define the term radius of relative stiffness l in cm is given by the equation

$$l = \sqrt[4]{\frac{Eh^3}{12K(1-\mu^2)}}$$

where E = modulus of elasticity of cement concrete in kg/cm^2 (3.0×10^5)

μ = Poisson's ratio of concrete (0.15)

h = slab thickness in cm

K = modulus of sub-grade reaction kg/cm^3

1.9.3 Critical Load Positions

Since the pavement slab has finite length and width, either the character or the intensity of maximum stress induced by the application of a given traffic load is dependent on the location of the load on the pavement surface. There are three typical locations namely the interior, edge and corner, where differing conditions of slab continuity exist. These locations are termed as critical load position.

1.9.4 Equivalent Radius of Resisting Section

When the interior point is loaded, only a small area of the pavement is resisting the bending moment of the plate. Westergaard give a relation for equivalent radius of the resisting section in cm in the equation

$$b = \begin{cases} \sqrt{1.6a^2 + h^2} - 0.675h & \text{if} \quad a \le 1.724h \\ a & \text{otherwise} \end{cases}$$

a = radius of contact area of wheel

1.9.5 Wheel Load Stresses-westergaard's Stress Equation

The cement concrete slab is assumed to be homogeneous and to have uniform elastic properties with vertical sub-grade reaction being proportional to the deflection. Westergaard developed relationships for the stress at interior, edge and corner regions, denoted as s_i, s_e, s_c in kg/cm^2 respectively and given by the equation

$$\sigma_i = \frac{0.316P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right) + 1.069\right]$$

$$\sigma_e = \frac{0.572P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right)+0.359\right]$$

$$\sigma_c = \frac{3P}{h^2}\left[1-\left(\frac{a\sqrt{2}}{l}\right)^{0.6}\right]$$

where h = slab thickness in cm

P = wheel load in kg

a = radius of the wheel load distribution in cm

l = radius of the relative stiffness in cm

b = radius of the resisting section in cm

1.9.5.1 Temperature Stresses

Temperature stresses are developed in cement concrete pavement due to variation in slab temperature. This is caused by :

1. Daily variation resulting in a temperature gradient across the thickness of the slab
2. Seasonal variation resulting in overall change in the slab temperature the former result in warping stresses and the later in frictional stresses.

1.9.5.2 Warping Stress

The warping stress at the interior, edge and corner regions, denoted as $\sigma_{t_i}, \sigma_{t_e}, \sigma_{t_c}$ in kg/cm^2 respectively and given by the equation

$$\sigma_{t_i} = \frac{E\alpha t}{2}\left(\frac{C_x+\mu C_y}{1-\mu^2}\right)$$

$$\sigma_{t_e} = \text{Max}\left(\frac{C_x E\alpha t}{2}, \frac{C_y E\alpha t}{2}\right)$$

$$\sigma_{t_c} = \frac{E\alpha t}{3(1-\mu)}\sqrt{\frac{a}{l}}$$

where E = modulus of elasticity of concrete in kg/cm^2 (3×10^5)

α = thermal coefficient of concrete (12×10^{-6}/°C)

t = temperature difference between the top and bottom of the slab

C_x and C_y = coefficient based on L_x/l in the desired direction and L_y/l right angle to the desired direction

μ = Poisson's ratio (0.15)

a = radius of the contact area

l = radius of the relative stiffness

L_x = Spacing between transverse joint

L_y = Spacing between longitudinal joint.

1.9.5.3 Frictional Stresses

The frictional stress σ_f in kg/cm^2 is given by the equation

$$\sigma_f = \frac{\gamma L f}{2 \times 10^4}$$

where γ = unit weight of concrete in kg/m^3 (2400)

f = coefficient of sub grade friction (1.5)

L = length of the slab in meters

1.9.5.4 Combination of Stresses

The cumulative effect of the different stress give rise to the following critical cases

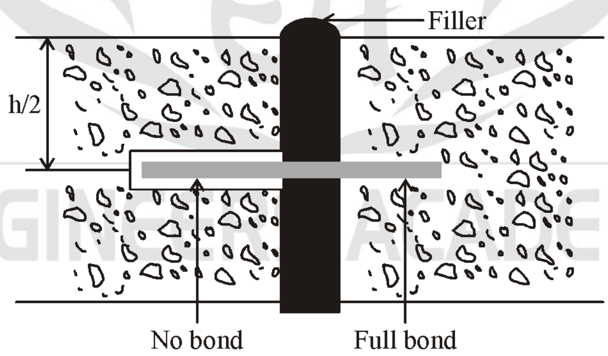
- **Summer, mid-day :** The critical stress is for edge region at bottom given by $\sigma_{critical} = \sigma_e + \sigma_{t_c} - \sigma_f$
- **Winter, mid-day :** The critical combination of stress is for the edge region at bottom given by $\sigma_{critical} = \sigma_e + \sigma_{t_c} + \sigma_f$
- **Summer mid-nights :** The critical combination of stress is for the corner region at top given by $\sigma_{critical} = \sigma_e + \sigma_{t_c}$

1.10 DESIGN OF JOINTS

1.10.1 Expansion Joints

The purpose of the expansion joint is to allow the expansion of the pavement due to rise in temperature with respect to construction temperature. The design consideration are :

- Provided normal to the longitudinal direction.
- Design involves finding the joint spacing for a given expansion joint thickness (say 2.5 cm specified by IRC) subjected to some maximum spacing (say 140 m as per IRC)



1.10.2 Contraction Joints

The purpose of the contraction joint is to allow the contraction of the slab due to fall in slab temperature below the construction temperature. The design considerations are :

- The movement is restricted by the sub-grade friction
- Design involves the length of the slab given by

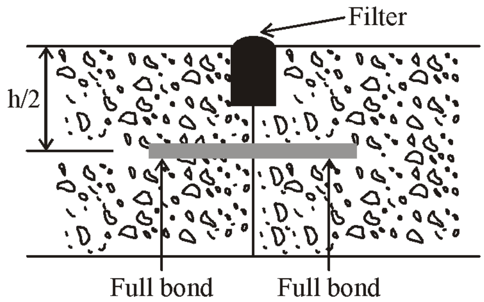
$$L_c = \frac{2 \times 10^4 S_c}{W.f}$$

where, S_c = allowable stress in tension in cement concrete and is taken as 0.8 kg/cm^2

W = unit weight of the concrete which can be taken as 2400 kg/m^3

f = coefficient of sub-grade friction which can be taken as 1.5

Steel reinforcements can be used, however with a maximum spacing of 4.5 m as per IRC.



Contraction joint

1.10.3 Dowel Bars

The purpose of the dowel bar is to effectively transfer the load between two concrete slabs and to keep the two slabs at same height. The dowel bars are provided in the direction of the traffic (longitudinal). The design considerations are

- Mild steel rounded bars
- Bonded on one side and free on other side

1.10.3.1 Bradbury's Analysis

Bradbury's analysis gives load transfer capacity of single dowel bar in shear, bending and bearing as follow

$$P_s = 0.785\ d^2 F_s$$

$$P_f = \frac{2d^3 F_f}{L_d + 8.8\delta}$$

$$P_b = \frac{F_b L_d^2 d}{12.5(L_d + 1.5\delta)}$$

where P = load transfer capacity of a single dowel bar in shears s, bending f and bearing b

d = diameter of the bar in cm

L_d = length of the embedment of dowel bar in cm

δ = joint width in cm

F_s, F_f, F_b = permissible stress in shear, bending and bearing for the dowel bar in kg/cm^2.

1.10.3.2 Design Procedure

Step-1: Find the length of the dowel bar embedded in slab L_d by equating $P_f = P_b$ i.e.,

$$L_d = 5d\sqrt{\frac{F_f(L_d + 1.5\delta)}{F_b(L_d + 8.8\delta)}}$$

δ = width of expansion joint

Step-2: Find the load transfer capacities P_s, P_s and P_b of single dowel bar with the L_d.

Step-3: Assume load capacity of dowel bar is 40 percent wheel load, find the load capacity factor f as

$$\max\left\{\frac{0.4P}{P_s}, \frac{0.4P}{P_f}, \frac{0.4P}{P_b}\right\}$$

Step-4: Spacing of the dowel bars. Effective distance upto which effective load transfer take place is given by 1.8 l, where l is the radius of relative stiffness.

- Assume a linear variation of capacity factor of 1.0 under load to 0 at 1.8 l.
- Assume a dowel spacing and find the capacity factor of the above spacing.
- Actual capacity factor should be greater than the required capacity factor.
- If not, do one more iteration with new spacing.

1.10.4 Tie Bars

In contrast to dowel bars, tie bars are not load transfer devices, but serve as a means to tie two slabs. Hence tie bars must be deformed or hooked and must be firmly anchored into the concrete to function properly. These are smaller then dowel bars and placed at large intervals. The are provided across longitudinal joints.

Step-1: **Diameter and spacing :** The diameter and the spacing is first found out by equating the total sub-grade friction and the total tensile stress for a unit length (one meter). Hence the area of steel per one meter in cm^2 is given by :

$$A_s \times S_s = b \times h \times W \times f$$

$$A_s = \frac{bhWf}{100S_s}$$

where
b = width of the pavement panel in m
h = depth of the pavement in cm
W = unit weight of the concrete (assume 2400 kg/m^3)
f = coefficient of friction (assume 1.5)
S_s = allowable working tensile stress in steel (assume 1750 kg/cm^2).

Assume 0.8 to 1.5 cm ϕ bars for the design.

Step-2: **Length of the tie bar :** Length of the tie bar is twice the length needed to develop bond stress equal to the working tensile stress and is given by

$$L_t = \frac{d\,S_s}{2\,S_b}$$

where
d = diameter of the bar
S_s = allowable tensile stress in kg/cm^2
S_b = allowable bond stress and can be assumed for plain and deformed bars respectively as 17.5 and 24.6 kg/cm^2.

1.11 TRAFFIC INTERSECTIONS

An intersection is where two or more roads join or cross. The operating efficiency of a highway and the safety there-of depend on the number and types of intersection en-route and the efficiency of the design of these intersections.

Intersections may be classified into two broad groups :

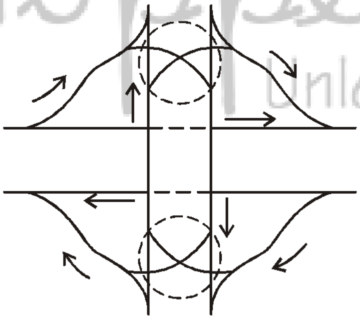
(i) Intersection at-grade : These include all roads which meet at more or less the same level. The traffic manoeuvres like merging, diverging and crossing are involved in the intersections at-grade. The intersections at-grade are further sub-classified into four categories, viz, (1) un-channelized (2) channelized (3) rotary intersection and (4) signalized intersection.

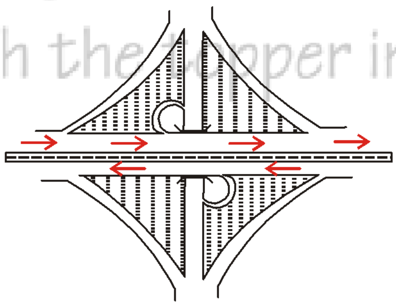
(ii) Grade separated intersection : The intersecting roads are separated by difference in level, thus eliminating the crossing manoeuvres. The grade separation is effected by taking one of the roads, say the major road either above the cross road by means of an over-pass or flyover or below the cross road by means of an under-pass. The turning movements between the grade separated cross roads are enabled by suitable 'interchange facilities'

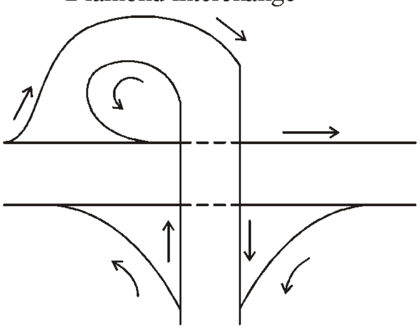
1.11.1 Interchange

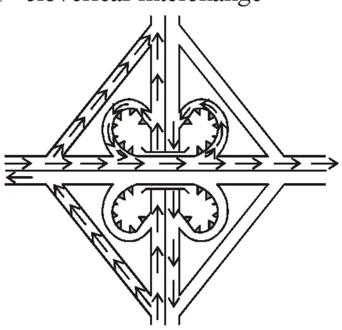
Interchange is a system where traffic between two or more roadways flows at different levels in the grade separated junctions. Different types are

1. **Diamond interchange :** It is the most popular form of four-leg interchange found in the urban locations where major and minor roads crosses. The important feature of this interchange is that it can be designed even if the major road is relatively narrow.
2. **Clover leaf interchange :** It is also a four leg interchange and is used when two highways of high volume and speed intersect each other. The main advantage of cloverleaf intersection is that it provides complete separation of traffic. Also high speed at intersections can be achieved. But the disadvantage is that large area of land is required.
3. **Trumpet interchange :** It is a three leg interchange. If one of the legs of the interchange meets a highway at some angle but does not cross it, then the interchange is called trumpet interchange.


Diamond interchange


Half cloverleaf interchange


Trumpet interchange


Cloverleaf interchange

1.11.2 Traffic Rotaries

Rotary intersections or round about are special form of at-grade intersections laid out for the movement of traffic in one direction around a central traffic island. Essentially all the major conflicts at an intersection namely the collision between through and right-turn movements are converted into milder conflicts namely merging and diverging. The vehicles entering the rotary are gently forced to move in a clockwise direction in orderly fashion. They then weave out of the rotary to the desired direction. Different shape of rotary are circular, elliptical, turbine and tengential.

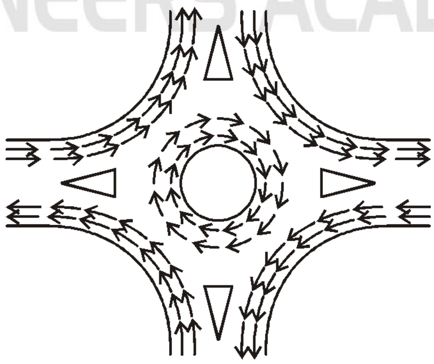
1.11.2.1 Guidelines for the Selection of Rotaries

Rotaries are not suitable for every location. There are few guidelines that help in deciding the suitability of a rotary. They are :

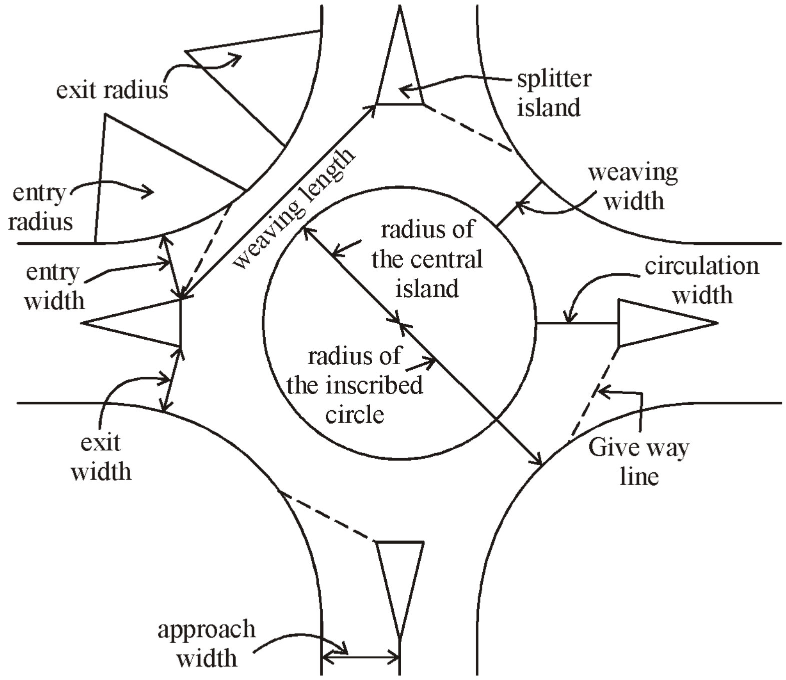
1. Rotaries are suitable when the traffic entering from all the four approaches are relatively equal.
2. A total volume of about 3000 vehicles per hour can be considered as the upper limiting case and a volume of 500 vehicles per hour is the lower limit.
3. A rotary is very beneficial when the proportion of the right-turn traffic is very high; typically it is suitable for more than 30 percent.
4. Rotaries are suitable when there are more than four approaches, or if there is no suitable lanes available for right-turn traffic.

1.11.2.2 Traffic Operations in a Rotary

1. **Diverging :** It is a traffic operation when the vehicles moving in one direction is separated into different streams according to their destinations.
2. **Merging :** Merging is the opposite of diverging, when traffic streams coming from various places and going to a common destination are joined together into a single stream it is referred to as merging.
3. **Weaving :** Weaving is the combined movement of both the merging and diverging movements in the same direction.



Traffic operations in a rotary



Design of a rotary

1.11.2.3 Width of The Rotary

The entry width and exit width of the rotary is governed by the traffic entering and leaving the intersection and the width of the approaching road. The width of the carriageway at entry and exit will be lower than the width of the carriageway at the approaches to enable reduction of speed. IRC suggests that a two lane road of 7 m width should be kept as 7 m for urban roads and 6.5 m for rural roads. Further, a three lane road of 10.5 m is to be reduced to 8 and 7.5 m respectively for urban and rural roads.

The width of the weaving section should be higher than the width at entry and exit. Normally this will be one lane more than the average entry and exit width. Thus weaving width is given as

$$w_{weaving} = \frac{(e_1 + e_2)}{2} + 3.5\,m$$

where e_1 = width of the carriageway at the entry

e_2 = carriageway width at exit

Weaving length determines how smoothly the traffic can merge and diverge. It is decided based on many factors such as weaving width, proportion of weaving traffic to the non-waving traffic etc. This can be best achieved by making the ratio of weaving length to the weaving width very high. A ratio of 4 is the minimum value suggested by IRC. Very large weaving length is also dangerous, as it may encourage over-speeding.

1.11.2.4 Capacity

The capacity of rotary is determined by the capacity of each weaving section. Transportation road research lab (TRL) proposed the following empirical formula to find the capacity of the weaving section.

$$Q_w = \frac{280w\left[1 + \frac{e}{w}\right]\left[1 - \frac{p}{3}\right]}{1 + \frac{w}{l}}$$

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

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

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

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

P = proportion of weaving traffic given by, $p = \frac{b+c}{a+b+c+d}$ in the range 0.4 to 1.0.

a = left turning traffic moving along left extreme lane

d = right turning traffic moving along right extreme lane

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

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

1.12 DESING OF PARKING FACILITY

Parking facilities may be broadly divided into two types:

(i) On – Street or kerb parking

(ii) Off – street parking

On – Street or kerb Parking:

In this type of parking, vehicles are parked along the kerb which may be designed for parking. kerb parking is quite convenient for those who could find a suitable space to park their vehicles near the place they wish to stop; but for others who could not find a parking space it is a problem and often they may have to park their vehicle at a far off place and walk down to the destination.

Different patterns of kerb parking are 'Parallel parking' and 'angle parking'.

Off – Street Parking:

At locations where the parking demand is high and kerb parking cannot be permitted in view of traffic congestion, off – street parking facilites are provided at the nearest locations depending on the availability of space for this purpose. When parking facility is provided at a separate place away from the road side or kerb, it is known as 'off – street parking'.

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