SYLLABUS

Subject Code: 10CV755 **IA Marks: 25**
No. of Lecture Hours/Week: 04 **Exam Hours: 03**
Total No. of Lecture Hours: 52 **Exam Marks: 100**

Unit - I

INTRODUCTION: Elements Geometric Design control factors like topography-design Sped design vehicle traffic –Capacity-Volume –Environment other factors-IRC and AAHO standards and specification- PCU concept for design. **06hrs**

Unit - II

CROSS SECTION ELEMENTS: pavement surface Characteristics Friction –skid and skid resistance- Pavement unevenness-light reflecting Characteristics- camber and its shapes – providing camber in the field Pavement width computation –kerbs and its types-Medians shoulders- foot paths- parking lanes-service roads-cycle tracks-driveways-guard rails- width of formation- Right of way –Design of Road humps as per IRC Specification **10hrs**

Unit – III

SIGHT DISTANCES: Important, types, Side distance at uncontrolled intersection, derivation, Factors affecting side distance, IRC, AASHTO standards, problems on above. **06hrs**

Unit - IV

HORIZONTAL ALIGNMENT: Definition, Checking the stability of vehicle, while moving on horizontal curve, Super elevation, Ruling minimum and maximum radius, Assumptions problems – method of providing super elevation for different curves – Extra widening of pavement on curves – objectives – Mechanical widening – psychological widening – Transition Curve – objectives – Ideal requirements – Types of transition curve – Method of evaluating length of transition curve – Setting the transition curve in the field, set back distance on horizontal curve and problems on above **08hrs**

Unit - V

VERTICAL ALIGNMENT: Gradient – Types of gradient – Design criteria of summit and valley curve – Design of vertical curves based on SSD – OSD – Night visibility considerations – Design standards for hilly roads – problems on the above. **05hr**s

Unit – VI

INTERSECTIONS DESIGN: Principle – At grade and Grade separated junctions – Types – channelization – Features of channelising Island – median opening – Gap in median at junction. **06hr**s

Unit - VII

ROTARY INTERSECTION: Elements – Advantages – Disadvantages – Design guide lines – problem on the above – Grade separated intersection – Three legged inter section – Diamond inter change – Half clover leaf – clover Leaf- Advantages- Disadvantages only **06hr**s

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Unit - VIII

HIGHWAY DRANIAGE Importance – sub surface drainage –surface drainage – Design of cross sections – Hydrological – Hydraulically considerations and design of filter media, problems on above. **05hr**s

TEST BOOKS:

1. Khanna S.K and Justo, C.E.O, Highway Engineering. Nem Chand and Bros.
2. L R Kadiyali., "Traffic Engineering and Transport Planning" Khanna Publishers.

REFERANCE BOOKS

1.Kadiyyali L.R., "Highway Engineering ‘’ Khanna Publishers.
2.Relevant IRC publications.
3.Papa coastas and Prevendours., ‘Transportation Engineering and Planning .,Phi,NewDelhi

CONTENT

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UNIT 1

INTRODUCTION

The geometric design of highways deals with the dimensions and layout of visible features of the highway. The emphasis of the geometric design is to address the requirement of the driver and the vehicle such as safety, comfort, efficiency, etc. The features normally considered are the cross section elements, sight distance consideration, horizontal curvature, gradients, and intersection. The design of these features is to a great extend influenced by driver behavior and psychology, vehicle characteristics, track characteristics such as speed and volume. Proper geometric design will help in the reduction of accidents and their severity. Therefore, the objective of geometric design is to provide optimum efficiency in track operation and maximum safety at reasonable cost. The geometric design of highway deals with the dimensions and layout of visible features of the highway such as alignment, cross section elements, sight distances, horizontal curvature, gradients, and intersections. The objective of geometric design is to provide maximum efficiency, in traffic operations with maximum safety at reasonable cost. The emphasis of the geometric design is to address the requirement of the driver and the vehicle such as safety, comfort, efficiency, etc. Geometric design of highways deals with the following elements:

1. Cross-section elements

2. Sight distance considerations

3. Horizontal alignment details

4. Vertical alignment details

5. Intersection elements

- **Cross-section elements** - It includes cross slope, various widths of road (i.e., width of pavement, formation width and road land width), surface characteristics and features in the road margins.
- **Sight distance considerations** - It the visible land ahead of the driver at horizontal and vertical curves and at intersections for the safe movements of vehicles.

- **Horizontal alignment** - Horizontal curves are introduced to change the direction of road. It includes features like super elevation, radius of curve, transition curve, extra widening and setback distance.
- **Vertical alignment** - Its components like gradient, vertical curves (i.e., summit curve and valley curve) sight distance and design of length of curves.
- **Intersection elements** - Proper design of intersection is very much essential for the safe and efficient traffic movements. Its features like layout, capacity, etc.

Factors affecting geometric design

Factors affecting the geometric designs are as follows.

- **Design speed**: Design speed is the single most important factor that affects the geometric design. It directly affects the sight distance, horizontal curve, and the length of vertical curves. Since the speed of vehicles vary with driver, terrain etc., a design speed is adopted for all the geometric design. Design speed is defined as the highest continuous speed at which individual vehicles can travel with safety on the highway when weather conditions are conducive. Design speed is different from the legal speed limit which is the speed limit imposed to curb a common tendency of drivers to travel beyond an accepted safe speed. Design speed is also different from the desired speed which is the maximum speed at which a driver would travel when unconstrained by either track or local geometry.
- **Topography**: It is easier to construct roads with required standards for a plain terrain. However, for a given design speed, the construction cost increases multi form with the gradient and the terrain
- Therefore, geometric design standards are different for different terrain to keep the cost of construction and time of construction under control. This is characterized by sharper curves and steeper gradients.
- **Traffic factors**: It is of crucial importance in highway design, is the traffic data both current and future estimates. Traffic volume indicates the level of services (LOS) for which the highway is being planned and directly affects the geometric features such as

width, alignment, grades etc., without traffic data it is very difficult to design any highway.

- **Design Hourly Volume and Capacity**: The general unit for measuring traffic on highway is the Annual Average Daily Traffic volume, abbreviated as AADT. The traffic flow (or) volume keeps fluctuating with time, from a low value during off peak hours to the highest value during the peak hour. It will be uneconomical to design the roadway facilities for the peak traffic flow. Therefore a reasonable value of traffic volume is decided for the design and this is called as Design Hourly Volume (DHV) which is determined from extensive traffic volume studies. The ratio of volume to capacity affects the level of service of the road. The geometric design is thus based on this design volume, capacity etc.
- **Environmental and other factors:** The environmental factors like air pollution, noise pollution, landscaping, aesthetics and other global conditions should be given due considerations in the geometric design of roads.

Passenger Car Unit (PCU):

Different classes of vehicles such as cars, vans, buses, trucks, auto rickshaw, motor cycles, pedal cycles etc. are found to use the common roadway facilities without segregation.

The flow of traffic with unrestricted mixing of different vehicle classes forms the ‘Mixed Traffic Flow’. In a mixed traffic condition, the traffic flow characteristics are very much complex when compared to homogeneous traffic consisting of passenger cars only. It is very difficult to estimate the traffic volume and capacity of roadway facilities under mixed traffic flow. Hence the different vehicle classes are converted to one common standard vehicle unit.

It is common practice to consider the passenger car as the standard vehicle unit to convert the other vehicle classes and this unit is called Passenger Car Unit (or) PCU. Thus in a mixed traffic flow, traffic volume and capacity are generally expressed as pcu / hr (or) pcu / lane/ hr and traffic density as pcu / km length of lane.

Factors affecting PCU Values:

- Vehicles characteristics such as dimensions, power, speed, acceleration and braking characteristics.
- Transverse and longitudinal gaps (or) clearances between moving vehicles which depends upon speed, driver characteristics.
- Traffic stream characteristics such as composition of different vehicle classes, mean speed and speed distribution of mixed traffic stream, volume to capacity ratio etc.
- Roadway characteristics such as road geometrics includes gradient, curve etc., rural or urban road, presence of intersections and the types of intersections.
- Regulation and control of traffic such as speed limit, one-way traffic, presence of different traffic control devices etc.
- Environmental and climatic conditions.

PCU values suggested by IRC:

Sl. no.	Vehicle class	PCU Values
1.	Passenger car, Auto rickshaw, Tempo, agricultural tractor	1.0
2.	Bus, Truck , agricultural tractor-tailer unit	3.0
3.	Motor cycle, scooter and pedal cycle	0.5
4.	Cycle rickshaw	1.5
5.	Horse drawn vehicles	4.0
6.	Small bullock cart and hand cart	6.0
7.	Large bullock cart	8.0

UNIT 2

CROSS SECTIONAL ELEMENTS

Pavement surface characteristics

For safe and comfortable driving four aspects of the pavement surface are important; the friction between the wheels and the pavement surface, smoothness of the road surface, the light reflection characteristics of the top of pavement surface, and drainage to water

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 affect 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.

Various factors that affect friction are:

- Type of the pavement (like bituminous, concrete, or gravel),
- Condition of the pavement (dry or wet, hot or cold, etc),
- Condition of the tyre (new or old), and
- Speed and load of the vehicle.

The frictional force that develops between the wheel and the pavement is the load acting multiplied by a factor called the coefficient of friction and denoted as f. The choice of the value of f is a very complicated issue since it depends on many variables. IRC suggests the coefficient of longitudinal friction as 0.35-0.4 depending on the speed and coefficient of lateral friction as 0.15. The former is useful in sight distance calculation and the latter in horizontal curve design.

Unevenness

It is always desirable to have an even surface, but it is seldom possible to have such a one. Even if a road is constructed with high quality pavers, it is possible to develop unevenness due to pavement failures. Unevenness affects the vehicle operating cost, speed, riding comfort, safety, fuel consumption and wear and tear of tyres. Unevenness index is a measure of unevenness which is the cumulative measure of vertical undulations of the pavement surface recorded per unit horizontal length of the road. An unevenness index value less than 1500 mm/km is considered as good, a value less than 2500 mm.km is satisfactory up to speed of 100 kmph and values greater than 3200 mm/km is considered as uncomfortable even for 55 kmph.

Light reflection characteristics

- White roads have good visibility at night, but caused glare during day time.
- Black roads has no glare during day, but has poor visibility at night
- Concrete roads has better visibility and less glare

It is necessary that the road surface should be visible at night and reflection of light is the factor that answers it.

Drainage

The pavement surface should be absolutely impermeable to prevent seepage of water into the pavement layers. Further, both the geometry and texture of pavement surface should help in draining out the water from the surface in less time.

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

Too steep slope is undesirable for it will erode the surface. Camber is measured in 1 in n or n% (Eg. 1 in 50 or 2%) and the value depends on the type of pavement surface. The values suggested by IRC for various categories of pavement is given in Table below .The common types of camber are parabolic, straight, or combination of them .

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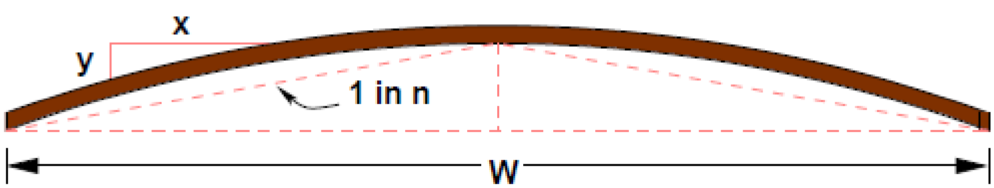
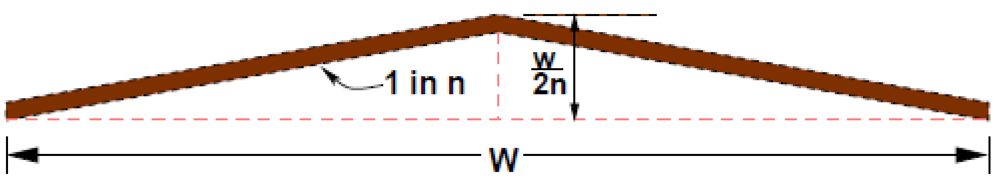
Table 12:1: IRC Values for camber

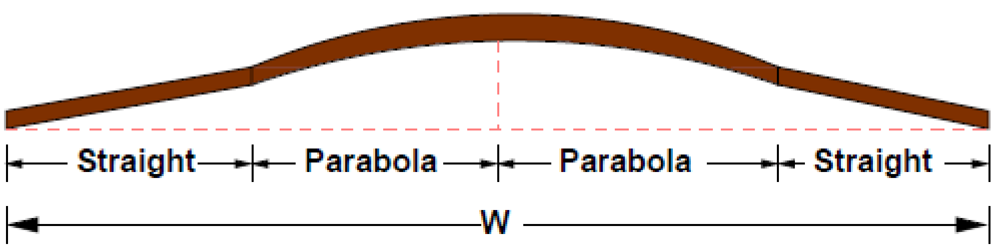
Surface type	Heavy rain	Light rain
Concrete/Bituminous	2 %	1.7 %
Gravel/WBM	3 %	2.5 %
Earthen	4 %	3.0 %



a. Parabolic camber $y=2x^2/nw$



b. Straight line camber

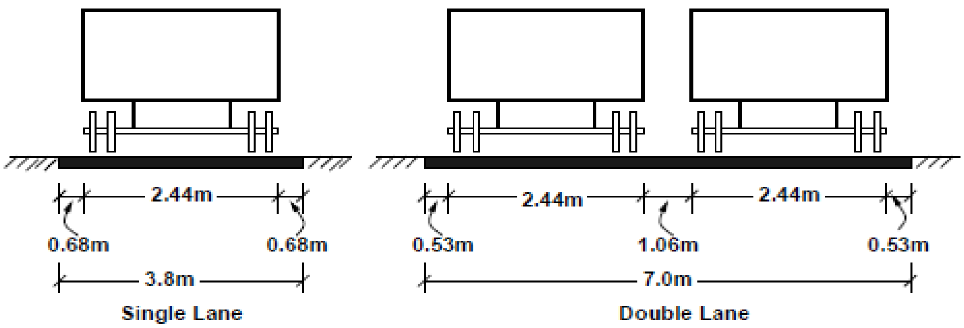


c. Combination of straight and parabolic camber

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 and the desirable side clearance for single lane traffic is 0.68 m. This requires minimum of lane width of 3.75 m for a single lane road However, the side clearance required is about 0.53 m, on both side and 1.06 m in the center. Therefore, a two lane road require minimum of 3.5 meter for each lane .The desirable carriage way width recommended by IRC is given in Table below.

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

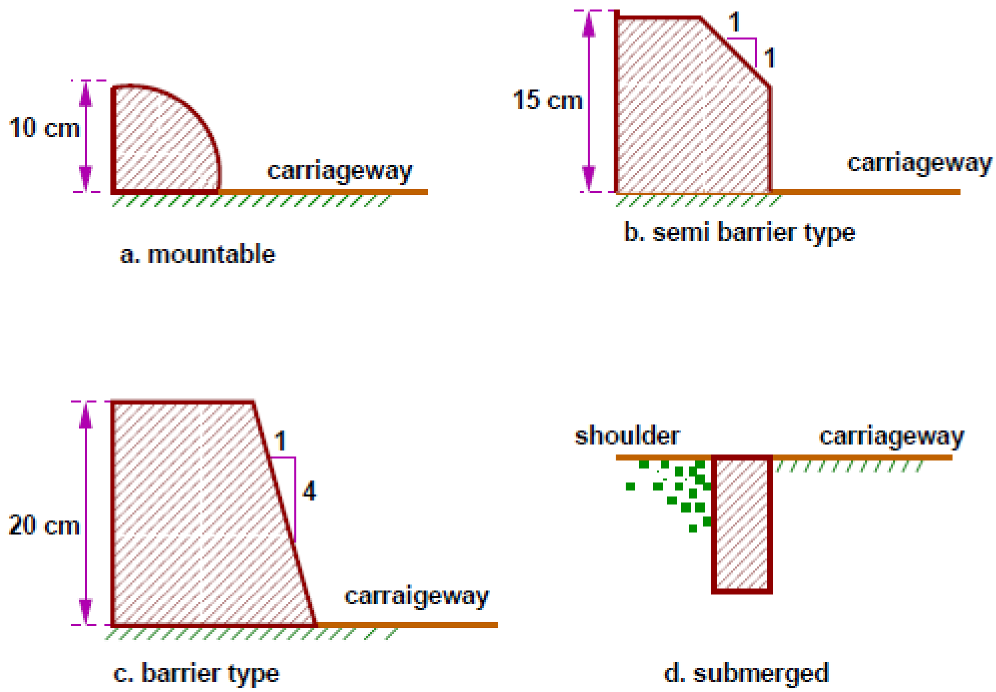


Kerbs

Kerbs indicate the boundary between the carriage way and the shoulder or islands or footpaths.

- **Low or mountable kerbs**: This type of kerbs is provided such that they encourage the traffic to remain in the through traffic lanes and also allow the driver to enter the shoulder area with little difficulty. The height of this kerb is about 10 cm above the pavement edge with a slope which allows the vehicle to climb easily. This is usually provided at medians and channelization schemes and also helps in longitudinal drainage.
- **Semi-barrier type kerbs**: When the pedestrian traffic is high, these kerbs are provided. Their height is 15 cm above the pavement edge. This type of kerb prevents encroachment of parking vehicles, but at acute emergency it is possible to drive over this kerb with some difficulty.
- **Barrier type kerbs**: They are designed to discourage vehicles from leaving the pavement. They are provided when there is considerable amount of pedestrian traffic. They are placed at a height of 20 cm above the pavement edge with a steep batter.

➢ **Submerged kerbs**: They are used in rural roads. The kerbs are provided at pavement edges between the pavement edge and shoulders. They provide lateral confinement and stability to the pavement.



Road margins

The portion of the road beyond the carriageway and on the roadway can be generally called road margin.

Various elements that form the road margins are given below.

➢ **Shoulders**: Shoulders are provided along the road edge and is intended for accommodation of stopped vehicles, serve as an emergency lane for vehicles and provide lateral support for base and surface courses. The shoulder should be strong enough to bear the weight of a fully loaded truck even in wet conditions. The shoulder width should be adequate for giving working space around a stopped vehicle. It is desirable to have a width of 4.6 m for the shoulders. A minimum width of 2.5 m is recommended for 2-lane rural highways in India.

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- **Parking lanes:** Parking lanes are provided in urban lanes for side parking. Parallel parking is preferred because it is safe for the vehicles moving on the road. The parking lane should have a minimum of 3.0 m width in the case of parallel parking.
- **Bus-bays:** Bus bays are provided by recessing the kerbs for bus stops. They are provided so that they do not obstruct the movement of vehicles in the carriage way. They should be at least 75 meters away from the intersection so that the traffic c near the intersections is not affected by the bus-bay.
- **Service roads:** Service roads or frontage roads give access to access controlled highways like freeways and expressways. They run parallel to the highway and will be usually isolated by a separator and access to the highway will be provided only at selected points. These roads are provided to avoid congestion in the expressways and also the speed of the traffic in those lanes is not reduced.
- **Cycle track:** Cycle tracks are provided in urban areas when the volume of cycle traffic is high Minimum width of 2 meter is required, which may be increased by 1 meter for every additional track.
- **Footpath:** Footpaths are exclusive right of way to pedestrians, especially in urban areas. They are provided for the safety of the pedestrians when both the pedestrian traffic and vehicular traffic is high. Minimum width is 1.5 meter and may be increased based on the traffic. The footpath should be either as smooth as the pavement or smoother than that to induce the pedestrian to use the footpath.

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 Table

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

Right of way

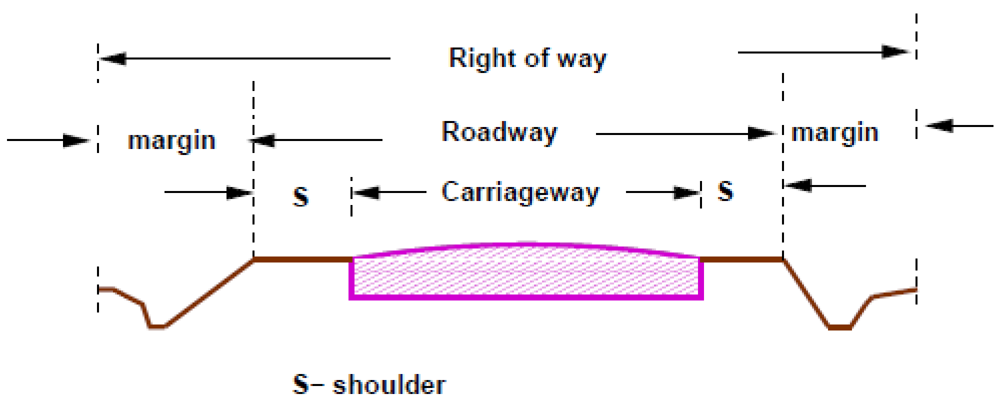
Right of way (ROW) or land width is the width of land acquired for the road, along its alignment. It should be adequate to accommodate all the cross-sectional elements of the highway and may reasonably provide for future development. To prevent ribbon development along highways, control lines and building 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 and their 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.

Road classification	Roadway width in m	
	Plain and rolling terrain	Mountainous and steep terrain
Open areas		
NH/SH	45	24
MDR	25	18
ODR	15	15
VR	12	9
Built-up areas		
NH/SH	30	20
MDR	20	15
ODR	15	12
VR	10	9

The importance of reserved land is emphasized by the following. Extra width of land is available for the construction of roadside facilities. Land acquisition is not possible later, because the land may be occupied for various other purposes (buildings, business etc.) The normal ROW requirements for built up and open areas as specified by IRC is given in table above. A typical cross section of a ROW is given in figure below.



Design the road hump as per IRC recommendations

These are round shaped elevated surface formed across roadways as physical devices to reduce the speed of the vehicles on minor or secondary or unimportant uncontrolled roads

The road humps design may be carried out based upon the finding of field experiment. They are observed that the ratio of cross sectional area to width of the road hump is significant for controlling hump cross speed, this is given the equation

$$1/V85 = 0.0212 + 0.4072(A/W)$$

Where,

V85 = Desired design 85th percentile of the hump crossing speed

A/W = Area to width ratio

UNIT 3

SIGHT DISTANCE

The safe and efficient operation of vehicles on the road depends very much on the visibility of the road ahead of the driver. Thus the geometric design of the road should be done such that any obstruction on the road length could be visible to the driver from some distance ahead. This distance is said to be the sight distance.

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 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 lights
- Safe sight distance to enter into an intersection.

The most important consideration in all these is that at all times the driver traveling at the design speed of the highway must have sufficient carriageway distance within his line of vision to allow him to stop his vehicle before colliding with a slowly moving or stationary object appearing suddenly in his own traffic lane.

The computation of sight distance depends on:

- **Reaction time of the driver**

Reaction time of a driver is the time taken from the instant the object is visible to the driver to the instant when the brakes are applied. The total reaction time may be split up into four components based on PIEV theory. In practice, all these times are usually combined into a total perception-reaction time suitable for design purposes as well as for easy measurement. Many of the studies show that drivers require about 1.5 to 2 secs under normal conditions. However, taking into consideration the variability of driver characteristics, a higher value is normally used in design. For example, IRC suggests a reaction time of 2.5 secs.

- **Speed of the vehicle**

The speed of the vehicle very much affects the sight distance. Higher the speed, more time will be required to stop the vehicle. Hence it is evident that, as the speed increases, sight distance also increases.

- **Efficiency of brakes**

The efficiency of the brakes depends upon the age of the vehicle, vehicle characteristics etc. If the brake efficiency is 100%, the vehicle will stop the moment the brakes are applied. But practically, it is not possible to achieve 100% brake efficiency. Therefore the sight distance required will be more when the efficiency of brakes are less. Also for safe geometric design, we assume that the vehicles have only 50% brake efficiency.

- **Frictional resistance between the tyre and the road**

The frictional resistance between the tyre and road plays an important role to bring the vehicle to stop. When the frictional resistance is more, the vehicles stop immediately. Thus sight required will be less. No separate provision for brake efficiency is provided while computing the sight distance. This is taken into account along with the factor of longitudinal friction. IRC has specified the value of longitudinal friction in between 0.35 to 0.4.

- **Gradient of the road.**

Gradient of the road also affects the sight distance. While climbing up a gradient, the vehicle can stop immediately. Therefore sight distance required is less. While descending a gradient, gravity also comes into action and more time will be required to stop the vehicle. Sight distance required will be more in this case.

Stopping sight distance

Stopping sight distance (SSD) is the minimum sight distance available on a highway at any spot having sufficient length to enable the driver to stop a vehicle traveling at design speed, safely without collision with any other obstruction.

There is a term called safe stopping distance and is one of the important measures in traffic engineering. It is the distance a vehicle travels from the point at which a situation is first perceived to the time the deceleration is complete. Drivers must have adequate time if they are to suddenly respond to a situation. Thus in highway design, sight distance atleast equal to the safe stopping distance should be provided. The stopping sight distance is the sum of lag distance and the braking distance. Lag distance is the distance the vehicle traveled during the reaction time t and is given by **vt**, where v is the velocity in m/sec^2. Braking distance is the distance traveled by the vehicle during braking operation. For a level road this is obtained by equating the work done in stopping the vehicle and the kinetic energy of the vehicle. If F is the maximum frictional force developed and the braking distance is l, then work done against friction in stopping the vehicle is **F_l = fWl** where W is the total weight of the vehicle. The kinetic energy at the design speed is

$$\frac{1}{2}mv^2 = \frac{1}{2}\frac{Wv^2}{g}$$

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

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

Therefore, the SSD = lag distance + braking distance and given by:

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

Where, v is the design speed in m=sec2

t is the reaction time in sec,

g is the acceleration due to gravity and f is the coefficient of friction.

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

When there is an ascending gradient of say +n%, the component of gravity adds to braking action and hence braking distance is decreased. The component of gravity acting parallel to the surface which adds to the braking force is equal to

$$W \sin\alpha \approx W \tan\alpha = Wn/100$$

Equating kinetic energy and work done

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

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

Similarly the braking distance can be derived for a descending gradient. Therefore the general equation is given

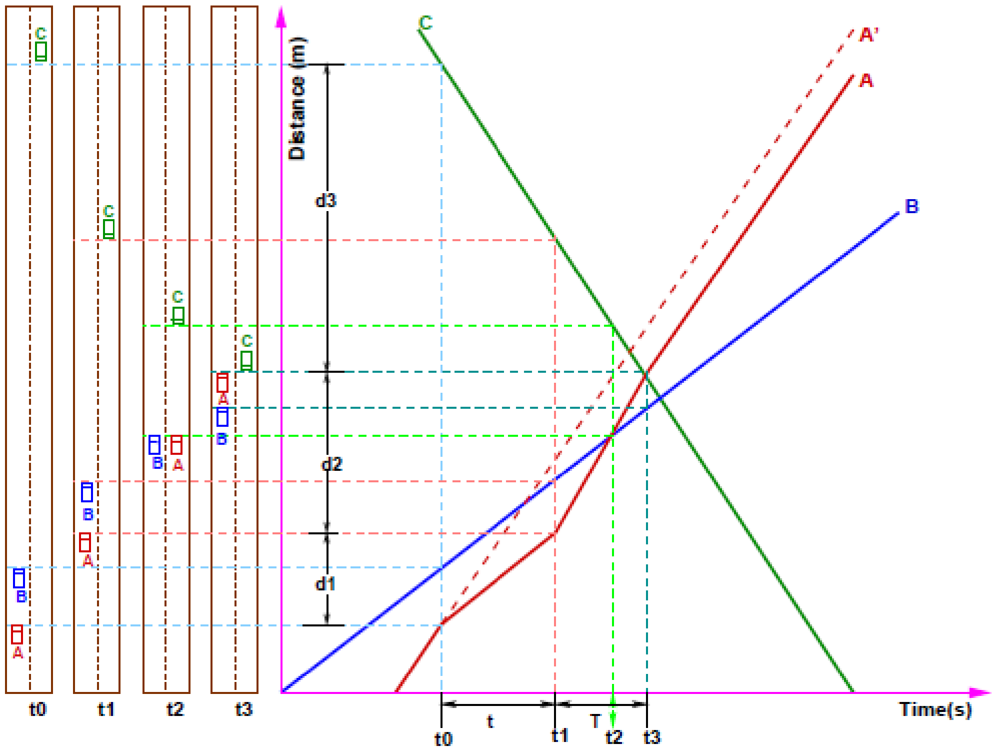
$$SSD = vt + \frac{v^2}{2g(f \pm 0.01n)}$$

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 1.2m 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



Time-space diagram: Illustration of overtaking sight distance

The dynamics of the overtaking operation is given in the figure which is a time-space diagram. The x-axis denotes the time and y-axis shows the distance traveled by the vehicles. The trajectory of the slow moving vehicle (B) is shown as a straight line which indicates that it is traveling at a constant speed. A fast moving vehicle (A) is traveling behind the vehicle B. The trajectory of the vehicle is shown initially with a steeper slope. The dotted line indicates the path of the vehicle A if B was absent. The vehicle A slows down to follow the vehicle B as shown in the figure with same slope from t_0 to t_1. Then it overtakes the vehicle B and occupies the left lane at time t3. The time duration T = t3 to t1 is the actual duration of the overtaking operation. The snapshots of the road at time t0; t1, and t3 are shown on the left side of the figure. From the Figure the overtaking sight distance consists of three parts.

- D_1 the distance traveled by overtaking vehicle A during the reaction time $t = t_1 - t_0$
- D_2 the distance traveled by the vehicle during the actual overtaking operation $T = t_3 - t_1$
- D_3 is the distance traveled by on-coming vehicle C during the overtaking operation (T).

$$OSD = d_1 + d_2 + d_3$$

It is assumed that the vehicle A is forced to reduce its speed to V_b the speed of the slow moving vehicle B and travels behind it during the reaction time t of the driver. So d_1 is given by:

$$d_1 = v_b t$$

Then the vehicle A starts to accelerate, shifts the lane, overtake and shift back to the original lane. The vehicle A maintains the spacing s before and after overtaking. The spacing s in m is given by:

$$s = 0.7v_b + 6$$

Let T be the duration of actual overtaking. The distance traveled by B during the overtaking operation is 2s+ V_b T. Also, during this time, vehicle A accelerated from initial velocity V_b and overtaking is completed while reaching final velocity v. Hence the distance traveled is given by:

$$\begin{aligned}
d_2 &= v_b T + \frac{1}{2}aT^2 \\
2s + v_b T &= v_b T + \frac{1}{2}aT^2 \\
2s &= \frac{1}{2}aT^2 \\
T &= \sqrt{\frac{4s}{a}} \\
d_2 &= 2s + v_b\sqrt{\frac{4s}{a}}
\end{aligned}$$

The distance traveled by the vehicle C moving at design speed v m/sec during overtaking operation is given by

$$d_3 = vT$$

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The overtaking sight distance is

$$OSD = v_b t + 2s + v_b \sqrt{\frac{4s}{a}} + vT$$

Where

v_b is the velocity of the slow moving vehicle in m/sec,

t the reaction time of the driver in sec,

s is the spacing between the two vehicles in m given by equation

a is the overtaking vehicles acceleration in m/sec^2

In case the speed of the overtaken vehicle is not given, it can be assumed that it moves 16 kmph slower the design speed.

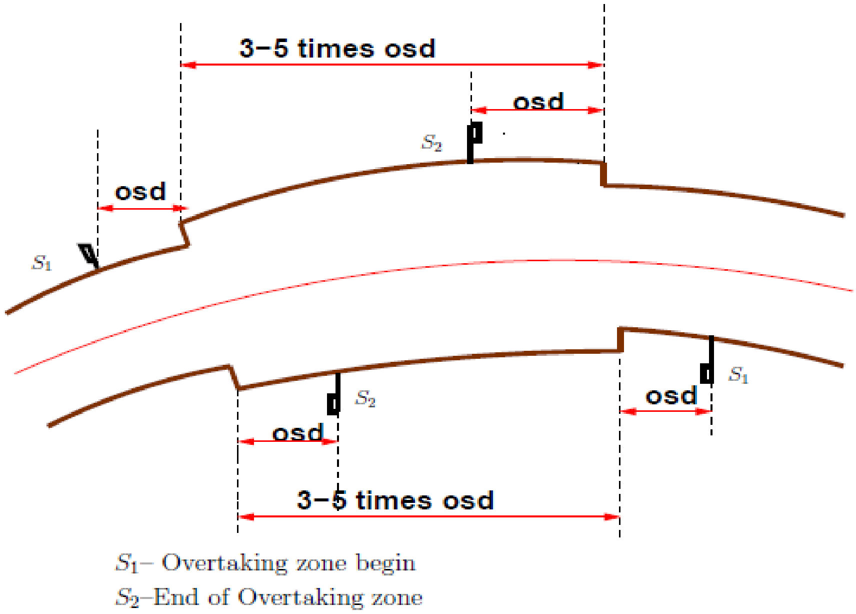
The acceleration values of the fast vehicle depends on its speed and given in Table below

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

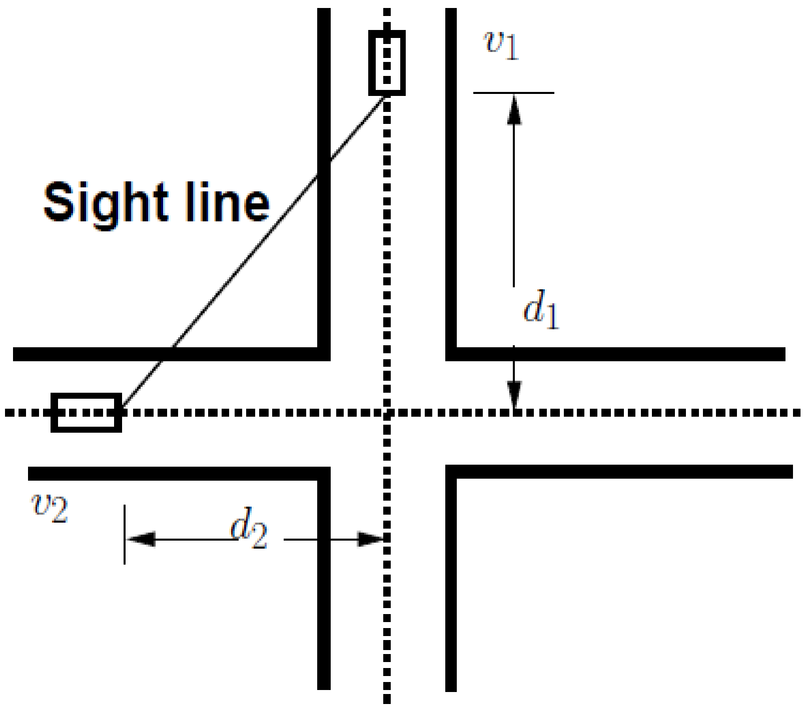
- On divided highways, d3 need not be considered
- On divided highways with four or more lanes, IRC suggests that it is not necessary to provide the OSD, but only SSD is sufficient.

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 5 time OSD and the minimum is three times OSD



Overtaking zones



Sight distance at intersections

Sight distance at intersections

At intersections where two or more roads meet, visibility should be provided for the drivers approaching the intersection from either side. 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. This is illustrated in the figure above. Design of sight distance at intersections may be used on three possible conditions:

- Enabling approaching vehicle to change the speed
- Enabling approaching vehicle to stop
- Enabling stopped vehicle to cross a main road

Problems

1. Calculate SSD for V =50kmph for (a) two-way traffic in a two lane road (b) two-way traffic in single lane road. (Hint: f=0.37, t=2.5) [Ans: (a)61.4 m (b) 122.8 m]

   Given: V =50km/hr = 13.9m/s f=0.37 t= 2.5 sec stopping distance=lag distance + braking distance

   $$SD = vt + v^2/2gf$$

   Stopping Distance = 61.4 m

   Stopping sight distance when there are two lanes = stopping distance= 61.4m.

   Stopping sight distance for a two way traffic for a single lane

   = 2[stopping distance]=122.8m

2. Find minimum sight distance to avoid head-on collision of two cars approaching at 90 kmph and 60 kmph. Given t=2.5sec, f=0.7 and brake efficiency of 50 percent in either case. (Hint: brake efficiency reduces the coefficient of friction by 50 percent).

   [Ans: SD=153.6+82.2=235.8m]

   Given: V1 =90 Km/hr. V2 = 60 Km/hr. t = 2.5sec. Braking efficiency=50%. f=.7.

   Stopping distance for one of the cars

   $$SD = vt + v^2/2gf$$

   Coefficient of friction due to braking efficiency of 50% = 0.5*0.7=0.35.

   Stopping sight distance of first car= SD1= 153.6m

   Stopping sight distance of second car= SD2= 82.2m

Stopping sight distance to avoid head on collision of the two approaching cars SD1+ SD2=235.8m.

3. Find SSD for a descending gradient of 2% for V=80kmph. [Ans: 132m].
   Given: Gradient (n) = -2V = 80 Km/hr.

$$SD = vt + v^2/2g(f - n\%)$$

   SSD on road with gradient = 132m.

4. Find head light sight distance and intermediate sight distance for V=65 kmph. (Hint: f=0.36, t=2.5 s,HSD=SSD, ISD=2*SSD) [Ans: 91.4 and 182.8 m]
   Given: V =65km/hr f=0.36 t= 2.5 sec

$$SD = vt + v^2/2g(f - n\%)$$

   Headlight Sight distance = 91.4m.
   Intermediate Sight distance= 2[SSD]= 182.8m

5. Overtaking and overtaken vehicles are at 70 and 40 kmph respectively. find (i) OSD (ii) min. and desirable length of overtaking zone (iii) show the sketch of overtaking zone with location of sign post (hint: a=0.99 m/sec2) [Ans: (i) 278 m (ii) 834 m/1390]

6. Calculate OSD for V=96 kmph. Assume all other data. (Hint: Vb=96-16kmph. a=0.72, t=2.5s) [Ans:OSD one way 342m, OSD two way 646m]

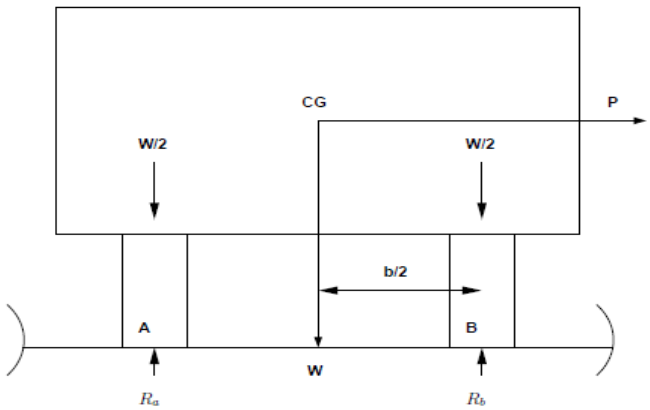
UNIT 4

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. In addition, it may increase the cost of vehicle operations and lower the highway capacity. Horizontal alignment design involves the understanding on the design aspects such as design speed and the effect of horizontal curve on the vehicles. The horizontal curve design elements include design of super elevation, extra widening at horizontal curves, design of transition curve, and set back distance.

Horizontal curve

The presence of horizontal curve imparts centrifugal force which is 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 overrun or to slide outward from the center 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 are illustrated in the figure



Effect of horizontal curve

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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. The centrifugal force P in kg\sqm is given by

$$P = \frac{Wv^2}{gR}$$

where

W is the weight of the vehicle in kg

v is the speed of the vehicle in m/sec,

g is the acceleration due to gravity in m/sec^2 and

R is the radius of the curve in m.

The centrifugal ratio or the impact factor P\W is given by

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

The centrifugal force has two effects: A tendency to overturn the vehicle about the outer wheels and a tendency for transverse skidding. Taking moments of the forces with respect to the outer wheel when the vehicle is just about to override

$$Ph = W\frac{b}{2} \quad \text{or} \quad \frac{\mathrm{P}}{\mathrm{W}} = \frac{\mathrm{b}}{2\mathrm{h}}$$

At the equilibrium over turning is possible when

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

And for safety the following condition must satisfy:

$$\frac{b}{2h} > \frac{v^2}{gR} \qquad \text{.............................. (1)}$$

The second tendency of the vehicle is for transverse skidding. i.e. when the centrifugal force P is greater than the maximum possible transverse skid resistance due to friction between the pavement surface and tyre. The transverse skid resistance (F) is given by:

$$\begin{aligned} F = & \quad F_A + F_B \\ = & \quad f(R_A + R_B) \\ = & \quad fW \end{aligned}$$

Where

- F_A and F_B is the fractional force at tyre A and B,
- R_A and R_B is the reaction at tyre A and B,
- f is the lateral coefficient of friction and
- W is the weight of the vehicle.

This is counteracted by the centrifugal force (P), and equating:

$$P = fW \quad \text{or} \quad \frac{\mathrm{P}}{\mathrm{W}} = \mathrm{f}$$

At equilibrium, when skidding takes place

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

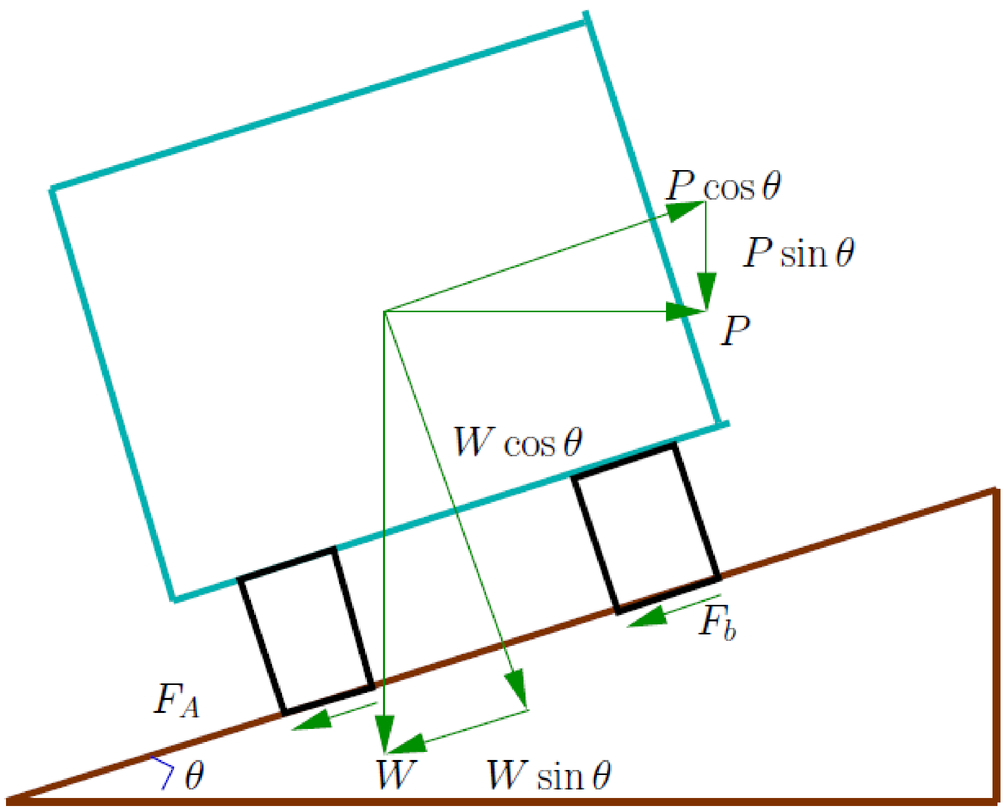
And for safety the following condition must satisfy

$$f > \frac{v^2}{gR} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. (2)$$

Equation (1) and (2) give the stable condition for design. If equation (1) is violated, the vehicles will overturn at the horizontal curve and if equation (2) is violated, the vehicle will skid at the horizontal curve.

Analysis of 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 curve 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 super elevation is shown in figure below



Forces acting on a vehicle on horizontal curve of radius R m at a speed of v m\sec^2 are:

- P the centrifugal force acting horizontally out-wards through the center of gravity,
- W the weight of the vehicle acting down-wards through the center of gravity, and
- F the friction force between the wheels and the pavement, along the surface inward.

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

$$\begin{aligned} P\cos\theta &= W\sin\theta + F_A + F_B \\ &= W\sin\theta + f(R_A + R_B) \\ &= W\sin\theta + f(W\cos\theta + P\sin\theta) \end{aligned}$$

where W is the weight of the vehicle, P is the centrifugal force, f is the coefficient of friction, θ is the transverse slope due to super elevation. Dividing by W cos θ, we get

$$\begin{aligned} \frac{P\cos\theta}{W\cos\theta} &= \frac{W\sin\theta}{W\cos\theta} + \frac{fW\cos\theta}{W\cos\theta} + \frac{fPsin\theta}{W\cos\theta} \\ \frac{P}{W} &= \tan\theta + f + f\frac{P}{W}\tan\theta \\ \frac{P}{W}(1 - f\tan\theta) &= \tan\theta + f \\ \frac{P}{W} &= \frac{\tan\theta + f}{1 - f\tan\theta} \end{aligned}$$

We have already derived an expression for P/W.By substituting this in equation above, we get

$$\frac{v^2}{gR} = \frac{\tan\theta + f}{1 - f\tan\theta}$$

This is an exact expression for super elevation. But normally, f=0.15 and θ< 4^0, 1-f tanθ and for small θ, tanθ approximately equal to sinθ=E\B=e, then the above equation becomes

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

Where,

e is the rate of super elevation,

f the coefficient of lateral friction 0.15,

v the speed of the vehicle in m\sec

R the radius of the curve in m and

g = 9:8m/sec^2

Three specific cases that can arise from equation above are as follows

- If there is no friction due to some practical reasons, then f = 0 and equation becomes

$$e = \frac{v^2}{gR}.$$

  This results in the situation where the pressure on the outer and inner wheels are same; requiring very high super-elevation e.

- If there is no super-elevation provided due to some practical reasons, then e = 0 and equation becomes

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

  This results in a very high coefficient of friction

- If e = 0 and f = 0:15 then for safe traveling speed from equation is given by

$$v_b = \sqrt{fgR}$$

  Where v_b is the restricted speed

Guidelines on super elevation

While designing the various elements of the road like superelevation, we design it for a particular vehicle called design vehicle which has some standard weight and dimensions. But in the actual case, the road has to cater for mixed traffic. Different vehicles with different dimensions and varying speeds ply on the road. For example, in the case of a heavily loaded truck with high centre of gravity and low speed, superelevation should be less; otherwise chances of toppling are more. Taking into practical considerations of all such situations, IRC has given some guidelines about the maximum and minimum superelevation etc. These are all discussed in detail in the following sections.

Design of super-elevation

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 f1 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,

$$v_a = \sqrt{0.22gR}$$

If $v_a \geq v$ design is adequate, otherwise use speeds adopt control measures or look for speed control measures

Maximum and minimum super-elevation

Depends on (a) slow moving vehicle and (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.

Attainment of super-elevation

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

(a) Rotating the outer edge about the crown: The outer half of the cross slopes 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 by: 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.

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, it is not desirable because re-alignment would be required if the design speed is increased in future. Therefore, a ruling minimum radius R_{ruling} can be derived by assuming maximum superelevation and coefficient of friction.

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

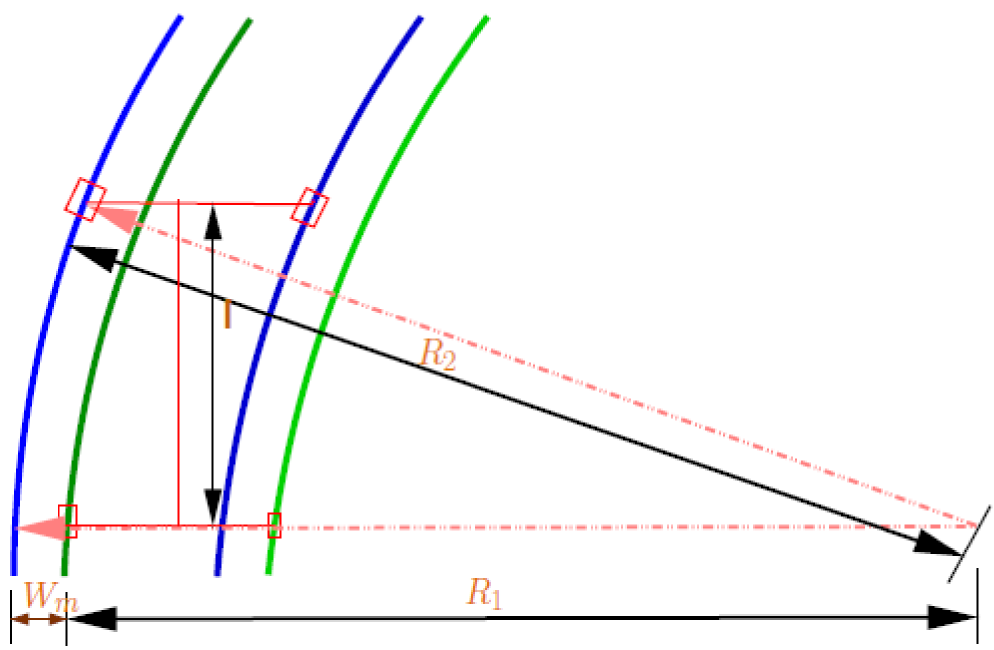
Ideally, the radius of the curve should be higher than R_{ruling}. However, very large curves are also not desirable. Setting out large curves in the field becomes difficult. In addition, it also enhances driving strain.

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. These are discussed in detail below.

Mechanical widening

The reasons for the mechanical widening are: When a vehicle negotiates a horizontal curve, the rear wheels follow a path of shorter radius than the front wheels as shown in figure 15.5. 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 traveling in opposite direction on curved roads as is provided on straight sections, there must be extra width of carriageway available. This is an important factor when high proportions of vehicles are using the road. Trailer trucks also need extra carriageway, depending on the type of joint. In addition speeds higher than the design speed causes transverse skidding which requires additional width for safety purpose. The expression for extra width can be derived from the simple geometry of a vehicle at a horizontal curve as shown in figure. Let R1 is the radius of the outer track line of the rear wheel, R_2 is the radius of the outer track line of the front wheel l is the distance between the front and rear wheel, n is the number of lanes, then the mechanical widening W_m (refer figure) is derived below:



Extra-widening at a horizontal curve

$$\begin{aligned} R_2^2 &= R_1^2 + l^2 \\ &= (R_2 - W_m)^2 + l^2 \\ &= R_2^2 - 2R_2W_m + W_m^2 + l^2 \\ 2R_2W_m - W_m^2 &= l^2 \end{aligned}$$

Therefore the widening needed for a single lane road is:

$$W_m = \frac{l^2}{2R_2 - W_m}$$

If the road has n lanes, the extra widening should be provided on each lane. Therefore, the extra widening of a road with n lanes is given by,

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

Please note that for large radius, R_2 is nearly equal R, which is the mean radius of the curve, then W_m is given by

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

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:

$$\begin{aligned} W_e &= W_m + W_{ps} \\ &= \frac{nl^2}{2R} + \frac{v}{2.64\sqrt{R}} \end{aligned}$$

Problems

1. A national highway passing through a rolling terrain has two horizontal curves of radius 450 m and 150 m. Design the required super-elevation for the curves as per IRC guidelines.

Solution: Assuming the ruling design speed for NH passing through a rolling terrain is 80 kmph. The coefficient of lateral friction f=0.15. The maximum permissible super elevation e=0.07.

Case 1: Radius = 450m

Step 1: Find e for 75 percent of design speed, neglecting f,

$$e_1 = \frac{(0.75v)^2}{gR}. \quad v = \frac{V}{3.6} = \frac{80}{3.6} = 22.22m/sec$$

$$e_1 = \frac{(0.75 \times 22.22)^2}{9.81 \times 450} = 0.0629$$

Step 2: $e_1 \leq$ 0:07. Hence the design is sufficient.

Answer: Design superelevation: 0.06.

Case 2: Radius = 150m

Step 1: Find e for 75 percent of design speed, neglecting f,

$$e_1 = \frac{(0.75v)^2}{gR}. \quad v = \frac{V}{3.6} = \frac{80}{3.6} = 22.22m/sec$$

$$e_1 = \frac{(0.75 \times 22.22)^2}{9.81 \times 150} = 0.188$$

Max.e to be provided = 0:07

Step 3: Find f1 for the design speed and max e, i.e.

$$f_1 = \frac{v^2}{gR} - e = \frac{22.22^2}{9.81 \times 150} - 0.07 = 0.265.$$

Step 4: Find the allowable speed v_a for the maximum e = 0:07 and f = 0:15

$$v_a = \sqrt{0.22gR} = \sqrt{0.22 \times 9.81 \times 150} =$$

$$17.99m/sec = 17.99 \times 3.6 = 64kmph$$

2. Given R=100m, V=50 kmph, f=0.15. Find:

(a) e if full lateral friction is assumed to develop [Ans: 0.047]

(b) Find f needed if no super elevation is provide [Ans: 0.197]

(c) Find equilibrium super-elevation if pressure on inner and outer wheel should be equal (Hint=0) [Ans: 0.197]

3. Two lane road, V=80 kmph, R=480 m, Width of the pavement at the horizontal curve=7.5 m. (i) Design super elevation for mixed traffic. (ii) By how much the outer edge of the pavement is to be raised with respect to the centerline, if the pavement is rotated with respect to centerline. [Ans :(i) 0.059 (ii) 0.22m]

4. Design rate of super elevation for a horizontal highway curve of radius 500 m and speed 100 kmph. [Ans: e=0.07, f=0.087 and within limits]

5. Given V=80 kmph, R=200m Design for super elevation. (Hint: f=0.15) [Ans: Allowable speed is 74.75 kmph and e=0.07]

6. 5.Calculate the ruling minimum and absolute minimum radius of horizontal curve of a NH in plain terrain. (Hint: V_{ruling}=100kmph, V_{min}=80kmph. e=0.07, f=0.15) [Ans: 360 and 230 m]

7. 6.Find the extra widening for W=7m, R=250m, longest wheel base, l=7m, V=70kmph. (Hint: n=2)[Ans: 0.662m]

8. 7.Find the width of a pavement on a horizontal curve for a new NH on rolling terrain. Assume all data. (Hint: V=80kmph for rolling terrain, normal W=7.0m, n=2, l=6.0m, e=0.07, f=0.15). [Ans: R_{ruling}=230m, W_e=0.71, W at HC=7.71m]

Horizontal 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
- To provide gradual introduction of extra widening.
- To enhance the aesthetic appearance of the road.

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 it fulfills the requirement of an ideal transition curve, that is;

- rate of change or centrifugal acceleration is consistent (smooth) and
- Radius of the transition curve is ∞ at the straight edge and changes to R at the curve point (Ls ∞ 1/R) and calculation and field implementation is very easy

Length of transition curve

The length of the transition curve should be determined as the maximum of the following three criteria: rate of change of centrifugal acceleration, rate of change of superelevation, and an empirical formula given by IRC.

- **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:

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

Therefore, the length of the transition curve L_{s1} 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:

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

Subject to:

c_{min} = 0:5

c_{max} = 0:8

- **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 60-150). Therefore, the length of the transition curve L_{s2} is:

$$L_{s_2} = Ne(W + W_e)$$

- **By empirical formula:**

IRC suggest the length of the transition curve is minimum for a plain and rolling terrain

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

For steep and hilly terrain is

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

And the shift s as

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

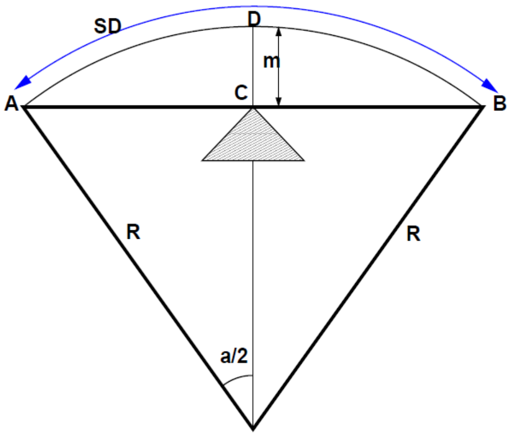
The length of the transition curve Ls is the maximum of equations above i.e,

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

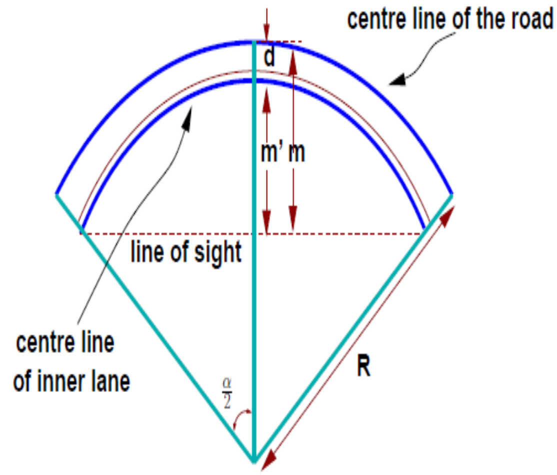
Setback Distance

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:

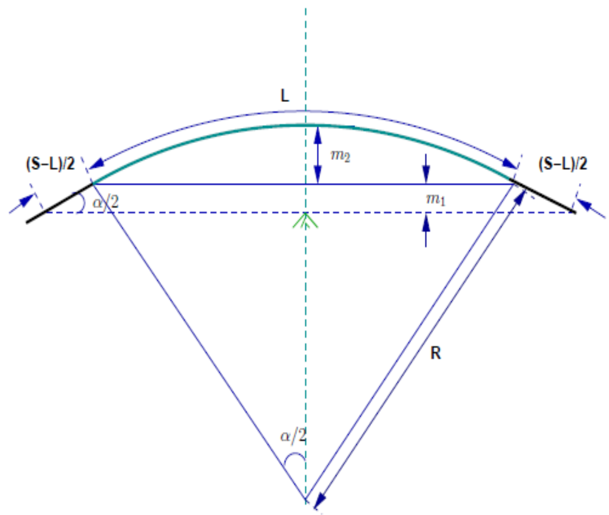
- Sight distance (OSD, ISD and OSD)
- Radius of the curve, and
- Length of the curve



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



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



Set-back for single lane roads ($L_s > L_c$)

Case (a) Ls < Lc

For single lane roads:

$$\begin{aligned} \alpha &= \frac{s}{R} \text{ radians} \\ &= \frac{180s}{\pi R} \text{ degrees} \\ \alpha/2 &= \frac{180s}{2\pi R} \text{ degrees} \end{aligned}$$

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

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

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

Case (b) Ls > Lc

For single lane:

$$\begin{aligned} m_1 &= R - R\cos(\alpha/2) \\ m_2 &= \frac{(S-L_c)}{2}\sin(\alpha/2) \end{aligned}$$

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

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

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Where,

$$\frac{\alpha}{2} = \frac{180L_c}{2\pi R}$$

For multi-lane road

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

m is given by

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

Problems

1. Calculate the length of transition curve and shift for V=65kmph, R=220m, rate of introduction of super elevation is 1 in 150, W+We=7.5 m. (Hint: c=0.57) [Ans: L_{s1}=47.1m,L_{s2}=39m (e=0.07, pav. rotatedw.r.t centerline), Ls3=51.9m, s=0.51m, Ls=52m]

2. NH passing through rolling terrain of heavy rainfall area, R=500m. Design length of Transition curve. (Hint: Heavy rainfall. Pavement surface rotated w.r.t to inner edge. V=80kmph, W=7.0m, N=1 in 150)[Ans: c=0.52, Ls1=42.3, Ls2=63.7m (e=0.057, W+We=7.45), Ls3=34.6m, Ls=64m]

3. Horizontal curve of R=400m, L=200 m. Compute setback distance required to provide

(a) SSD of 90m,

(b) OSD of 300 m.

Distance between center line of road and inner land (d) is 1.9m. [Ans: (a) α\2= 6:5^0, m=4.4 m (b) OSD>L, for multi-lane, with d=1.9, m=26.8 m]

UNIT 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 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.

Gradient

Gradient is the rate of rise or fall along the length of the road with respect to the horizontal. While aligning a highway, the gradient is decided for designing the vertical curve. Before finalizing the gradients, the construction cost, vehicular operation cost and the practical problems in the site also has to be considered. Usually steep gradients are avoided as far as possible because of the difficulty to climb and increase in the construction cost. More about gradients are discussed below.

Effect of gradient

The effect of long steep gradient on the vehicular speed is considerable. This is particularly important in roads where the proportion of heavy vehicles is significant. Due to restrictive sight distance at uphill gradients the speed of traffic is often controlled by these heavy vehicles. As a result, not only the operating costs of the vehicles are increased, but also capacity of the roads will have to be reduced. Further, due to high differential speed between heavy and light vehicles, and between uphill and downhill gradients, accidents abound in gradients.

Representation of gradient

The positive gradient or the ascending gradient is denoted as +n and the negative gradient as -n. The deviation angle N is: when two grades meet, the angle which measures the change of direction and is given by the algebraic difference between the two grades $(n_1 - (-n_2)) = n_1 + n_2 = \alpha_1+\alpha_2$

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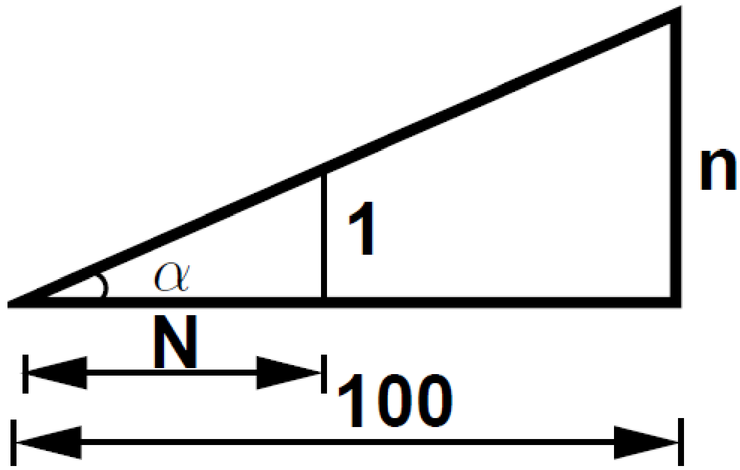
Example: 1 in 30 = 3.33% nearing to 2^0 is a steep gradient, while 1 in 50 = 2% nearing to $1^0 10'$ is a flatter gradient.



Representation of gradient

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

IRC Specifications for gradients for different roads

Types of gradient

Many studies have shown that gradient up to seven percent can have considerable effect on the speeds of the passenger cars. On the contrary, the speeds of the heavy vehicles are considerably reduced when long gradients as flat as two percent is adopted. Although, flatter gradients are desirable, it is evident that the cost of construction will also be very high. Therefore, IRC has specified the desirable gradients for each terrain. However, it may not be economically viable to adopt such gradients in certain locations; steeper gradients are permitted for short duration.

Ruling gradient, limiting gradient, exceptional gradient and minimum gradient are some types of gradients which are discussed below

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. In flatter terrain, it may be possible to provide at gradients, but in hilly terrain it is not economical and sometimes not possible also. 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. But our country has a heterogeneous traffic and hence it is not possible to lay down precise standards for the country as a whole. Hence IRC has recommended some values for ruling gradient for different types of terrain

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.

Exceptional gradient

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

Critical length of the grade

The maximum length of the ascending gradient which a loaded truck can operate without undue reduction in speed is called critical length of the grade. A speed of 25 kmph is a reasonable value. This value depends on the size, power, load, grad-ability of the truck, initial speed, final desirable minimum speed etc.

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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 requires 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. A minimum of 1 in 500 may be sufficient for concrete drain and 1 in 200 for open soil drains are found to give satisfactory performance.

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 which is intended to o set the extra tractive force involved at the curve. IRC gave the following specification for the grade compensation.

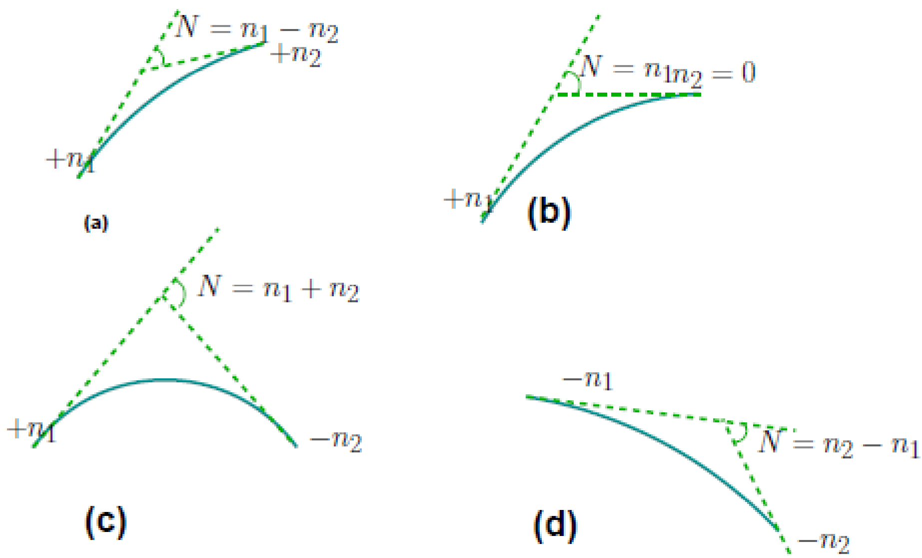
1. Grade compensation is not required for grades flatter than 4% because the loss of tractive force is negligible.
2. Grade compensation is (30+R)\R %, where R is the radius of the horizontal curve in meters
3. The maximum grade compensation is limited to 75\R %.

Summit curve

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

- when a positive gradient meets another positive gradient

- when positive gradient meets a flat gradient
- when an ascending gradient meets a descending gradient
- when a descending gradient meets another descending gradient



Types of summit curves

Type of Summit Curve

Many curve forms can be used with satisfactory results; the common practice has been to use parabolic curves in summit curves. This is primarily because of the ease with it can be laid out as well as allowing a comfortable transition from one gradient to another. Although a circular curve offers equal sight distance at every point on the curve, for very small deviation angles a circular curve and parabolic curves are almost congruent. Furthermore, the use of parabolic curves was found to give excellent riding comfort.

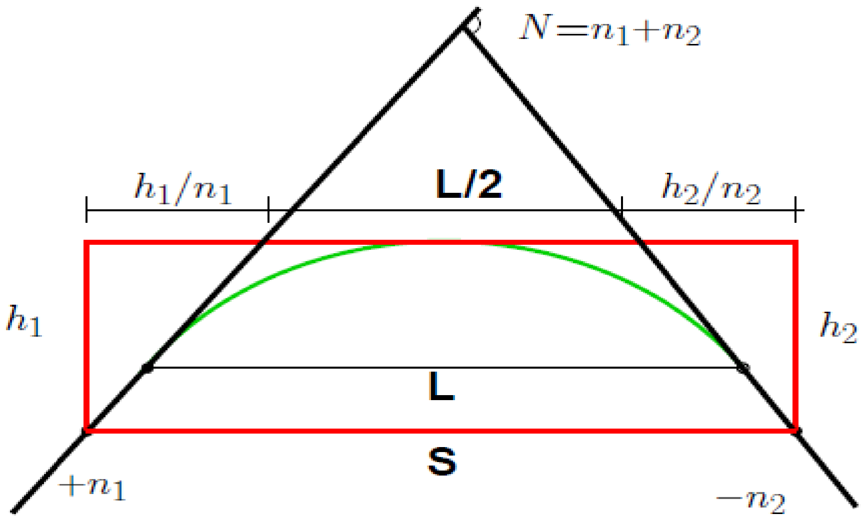
Design Consideration

In determining the type and length of the vertical curve, the design considerations are comfort and security of the driver, and the appearance of the profile alignment. Among these, sight distance requirements for the safety are most important on summit curves. The stopping sight distance or absolute minimum sight distance should be provided on these curves and where overtaking is not prohibited, overtaking sight distance or intermediate sight distance should be provided as far as possible. When a fast moving vehicle travels along a summit curve, there is less discomfort to the passengers. This is because the centrifugal force will be acting upwards while the vehicle negotiates a summit curve which is against the gravity and hence a part of the tyre pressure is relieved. Also if the curve is provided with adequate sight distance, the length would be sufficient to ease the shock due to change in gradient. Circular summit curves are identical since the radius remains same throughout and hence the sight distance. From this point of view, transition curves are not desirable since it has varying radius and so the sight distance will also vary. The deviation angle provided on summit curves for highways is very large, and so the simple parabola is almost congruent to a circular arc, between the same tangent points. Parabolic curves are easy for computation and also it had been found out that it provides good riding comfort to the drivers. It is also easy for field implementation. Due to all these reasons, a simple parabolic curve is preferred as summit curve.

Length of the summit curve

The important design aspect of the summit curve is the determination of the length of the curve which is parabolic. As noted earlier, the length of the curve is guided by the sight distance consideration. That is, a driver should be able to stop his vehicle safely if there is an obstruction on the other side of the road. Equation of the parabola is given by $y = ax^2$, where a = N \2L, where N is the deviation angle and L is the length of the In deriving the length of the curve, two situations can arise depending on the uphill and downhill gradients when the length of the curve is greater than the sight distance and the length of the curve is greater than the sight distance.

Let L is the length of the summit curve, S is the SSD/ISD/OSD, N is the deviation angle, h1 driver's eye height (1.2 m), and h2 the height of the obstruction, then the length of the summit curve can be derived for the following two cases. The length of the summit curve can be derived from the simple geometry as shown below:



Length of summit curve (L < S)

Case a. Length of summit curve greater than sight distance (L > S)

The situation when the sight distance is less than the length of the curve is shown in figure above

$$\begin{aligned} y &= ax^2 \\ a &= \frac{N}{2L} \\ h_1 &= aS_1^2 \\ h_2 &= aS_2^2 \\ S_1 &= \sqrt{\frac{h_1}{a}} \\ S_2 &= \sqrt{\frac{h_2}{a}} \end{aligned}$$

$$\begin{aligned} S_1 + S_2 &= \sqrt{\frac{h_1}{a}} + \sqrt{\frac{h_2}{a}} \\ S^2 &= \left(\frac{1}{\sqrt{a}}\right)^2 \left(\sqrt{h_1} + \sqrt{h_2}\right)^2 \end{aligned}$$

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

Case b. Length of summit curve less than sight distance

The second case is illustrated in figure above

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,

$$\frac{dS}{dh_1} = \frac{-h_1}{n_1^2}+\frac{h_2}{N-{n_1}^2} = 0 h_1\left(N-n_1\right)^2 = \quad h_2 n_1^2$$

$$h_1\left(N^2+n_1^2-2Nn_1\right) = \quad h_2 n_1^2$$
$$h_1N^2+h_1n_1^2-2Nn_1h_1 = h_2n_1^2$$
$$\left(h_2-h_1\right)n_1^2+2Nh_1n_1-h_1N^2 = 0$$

Solving the quadratic equation for n_1,

$$n_1 = \frac{-2Nh_1 \pm \sqrt{(2Nh_1)^2 - 4(h_2-h_1)(-h_1N^2)}}{2(h_2-h_1)}$$
$$= \frac{-2Nh_1 + \sqrt{4N^2h_1^2 + 4h_1N^2h_2 - 4h_1^2N^2}}{2(h_2-h_1)}$$
$$= \frac{-2Nh_1 + 2N\sqrt{h_1h_2}}{2(h_2-h_1)}$$
$$n_1 = \frac{N\sqrt{h_1h_2} - h_1N}{h_2-h_1}$$

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

$$S = \frac{L}{2} + \frac{h_1}{\frac{N\sqrt{h_1h_2}-Nh_1}{h_2-h_1}} + \frac{h_2}{N - \frac{N\sqrt{h_1h_2}-Nh_1}{h_2-h_1}}$$

Solving for L,

$$\begin{aligned}
&= \frac{L}{2} + \frac{h_1(h_2-h_1)}{N(\sqrt{h_1h_2}-h_1)} + \frac{h_2(h_2-h_1)}{Nh_2 - Nh_1 - N\sqrt{h_1h_2} + Nh_1} \\
&= \frac{L}{2} + \frac{h_1(h_2-h_1)}{N(\sqrt{h_1h_2}-h_1)} + \frac{h_2(h_2-h_1)}{N(h_2-\sqrt{h_1h_2})} \\
&= \frac{L}{2} + \frac{h_1(h_2-h_1)(h_2-\sqrt{h_1h_2}) + (h_2-h_1)h_2(\sqrt{h_1h_2}-h_1)}{N(\sqrt{h_1h_2}-h_1)(h_2-\sqrt{h_1h_2})} \\
&= \frac{L}{2} + \frac{(h_2-h_1)(h_1h_2 - h_1\sqrt{h_1h_2} + h_2\sqrt{h_1h_2} - h_1h_2)}{N(\sqrt{h_1h_2}-h_1)(h_2-\sqrt{h_1h_2})} \\
&= \frac{L}{2} + \frac{(h_2-h_1)(\sqrt{h_1h_2}(h_2-h_1))}{N(h_2\sqrt{h_1h_2} - h_1h_2 + h_1\sqrt{h_1h_2} - h_1h_2)} \\
&= \frac{L}{2} + \frac{(h_2-h_1)\sqrt{h_1h_2}(\sqrt{h_2}+\sqrt{h_1})(\sqrt{h_2}-\sqrt{h_1})}{N\sqrt{h_1h_2}(h_2 - 2\sqrt{h_1h_2} + h_2)} \\
&= \frac{L}{2} + \frac{(h_2-h_1)(\sqrt{h_2}+\sqrt{h_1})(\sqrt{h_2}-\sqrt{h_1})}{N(\sqrt{h_2}-\sqrt{h_1})^2}
\end{aligned}$$

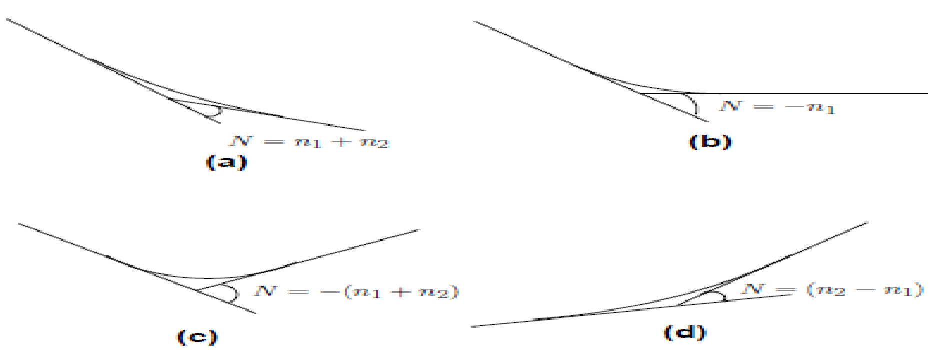
$$
\begin{aligned}
&= \frac{L}{2} + \frac{(h_2 - h_1)\left(\sqrt{h_2} + \sqrt{h_1}\right)}{N\left(\sqrt{h_2} - \sqrt{h_1}\right)} \\
&= \frac{L}{2} + \frac{\left(\sqrt{h_2} + \sqrt{h_1}\right)\left(\sqrt{h_2} - \sqrt{h_1}\right)\left(\sqrt{h_2} + \sqrt{h_1}\right)}{N\left(\sqrt{h_2} - \sqrt{h_1}\right)} \\
&= \frac{L}{2} + \frac{\left(\sqrt{h_2} + \sqrt{h_1}\right)^2}{N} \\
L &= 2S - \frac{2\left(\sqrt{h_2} + \sqrt{h_1}\right)^2}{N}
\end{aligned}
$$

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

When stopping sight distance is considered the height of driver's eye above the road surface (h_1) is taken as 1.2 metres, and height of object above the pavement surface (h_2) is taken as 0.15 metres. 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 metres.

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

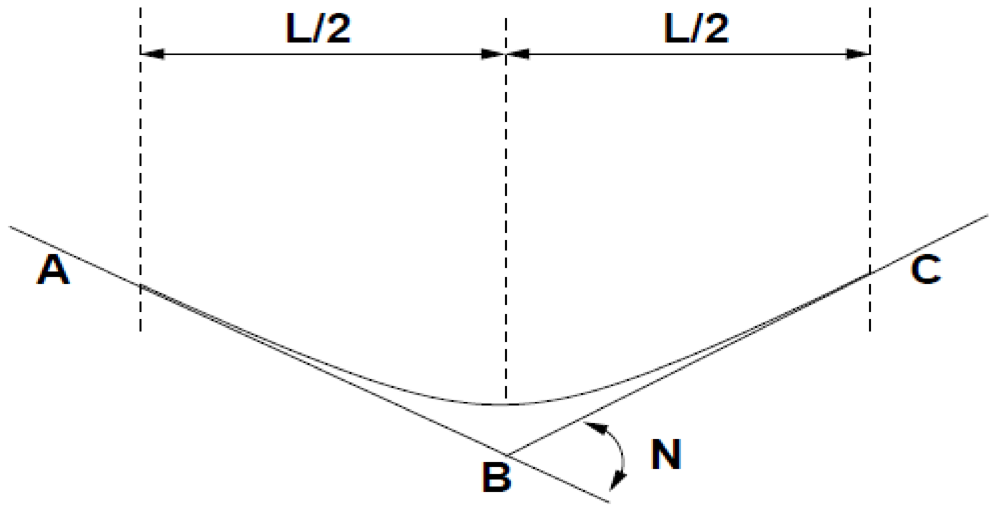


1. When a descending gradient meets another descending gradient.

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 gradient



Valley curve details

Design considerations

There is no restriction to sight distance at valley curves during day time. But visibility is reduced during night. In the absence or inadequacy of street light, the only source for visibility is with the help of headlights. Hence valley curves are designed taking into account of headlight distance. In valley curves, the centrifugal force will be acting downwards along with the weight of the vehicle, and hence impact to the vehicle will be more. This will result in jerking of the vehicle and cause discomfort to the passengers. Thus the most important design factors considered in valley curves are: (1) impact-free movement of vehicles at design speed and (2) availability of stopping sight distance under headlight of vehicles for night driving.

For gradually introducing and increasing the centrifugal force acting downwards, the best shape that could be given for a valley curve is a transition curve. Cubic parabola is generally preferred in vertical valley curves. See figure above during night, under headlight driving condition, sight distance reduces and availability of stopping sight distance under head light is very important. The head light sight distance should be at least equal to the stopping sight distance. There is no problem of overtaking sight distance at night since the other vehicles with headlights could be seen from a considerable distance.

Length of the valley curve

The valley curve is made fully transitional by providing two similar transition curves of equal length The transitional curve is set out by a cubic parabola $y = bx^3$ where $b = 2N/3L_2$ The length of the valley transition curve is designed based on two criteria:

1. Comfort criteria; that are allowable rate of change of centrifugal acceleration is limited to a comfortable level of about 0:6m\sec3.

2. Safety criteria; that is the driver should have adequate headlight sight distance at any part of the country.

Comfort criteria

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

$$\begin{aligned} c &= \frac{\frac{v^2}{R} - 0}{t} \\ &= \frac{\frac{v^2}{R} - 0}{\frac{L}{v}} \\ &= \frac{v^3}{LR} \\ L &= \frac{v^3}{cR} \end{aligned}$$

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

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

Therefore,

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

$$L_s = \sqrt[2]{\frac{Nv^3}{c}}$$

$$L = 2\sqrt[2]{\frac{Nv^3}{c}}$$

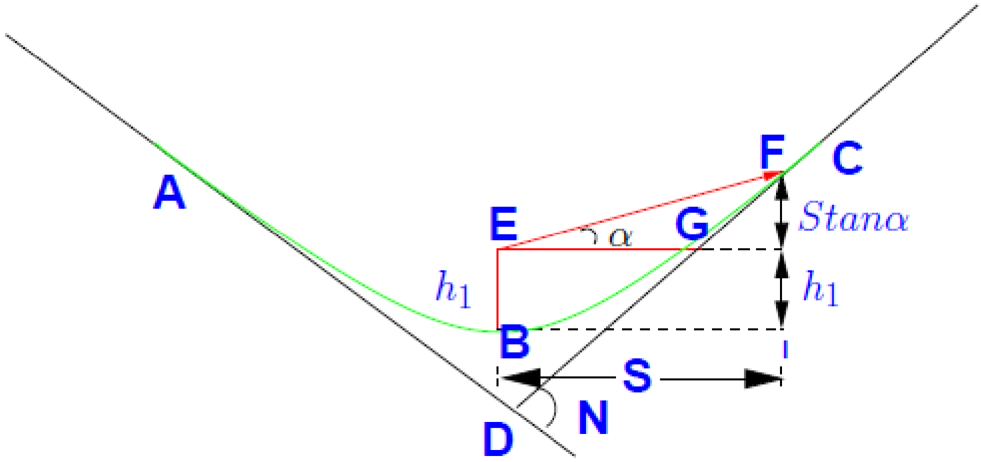
Where L is the total length of valley curve, N is the deviation angle in radians or tangent of the deviation angle or the algebraic difference in grades, and c is the allowable rate of change of centrifugal acceleration which may be taken as 0.6m\sec^3.

Safety criteria

Length of the valley curve for headlight distance may be determined for two conditions: (1) length of the valley curve greater than stopping sight distance and (2) length of the valley curve less than the stopping sight distance.

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 in infinite radius and the bottom of the curve will have minimum radius which is a property of the transition curve. The case is shown in figure below. From the geometry of the figure, we have

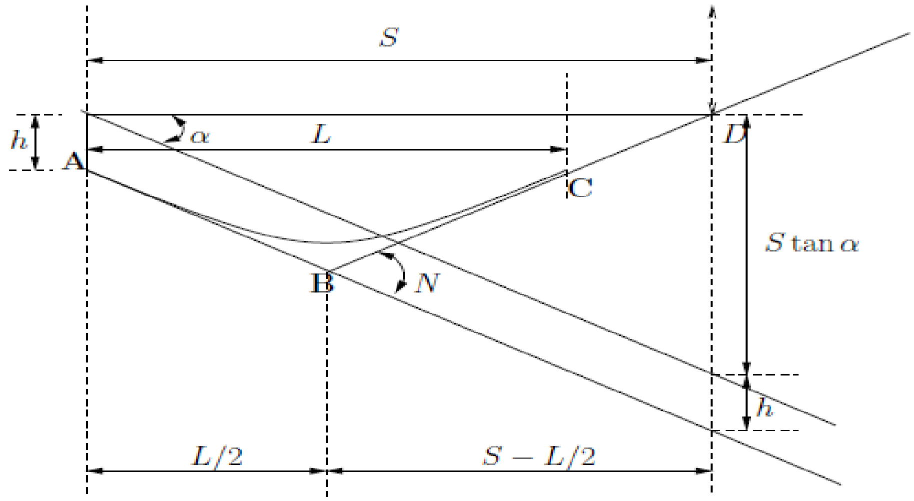


$$
\begin{aligned}
h_1 + S\tan\alpha &= aS^2 \\
&= \frac{NS^2}{2L} \\
L &= \frac{NS^2}{2h_1 + 2S\tan\alpha}
\end{aligned}
$$

Where N is the deviation angle in radians, h1 is the height of headlight beam, is the head beam inclination in degrees and S is the sight distance. The inclination α is almost equal to 1^0

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 figure below



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

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

Note that the above expression is approximate and is satisfactory because in practice, the gradients are very small and is acceptable for all practical purposes. We will not be able to know prior to which case to be adopted. Therefore both have to be calculated and the one which satisfies the condition is adopted.

UNIT 6

INTERSECTION

It is the most complex location on any highway. Conflicts are common at the intersections. This is because Vehicles moving in different direction want to occupy same space at the same time. In addition, the pedestrians also seek same space for crossing. Intersection is an area shared by two or more roads. It is some area designated for the vehicles to turn to different directions to reach their desired destinations. Its main function is to provide channelization of route direction. Drivers have to make split second decision at an intersection by considering this route. Intersection geometry, other vehicles, their speed, direction etc. A small error in judgment cans cause severe accidents. It also causes delay and it depends on type, geometry and type of control. Overall traffic flow depends on the performance of intersection. It also affects the capacity of the road. Especially in the case of an urban scenario, both from the accident perspective and the capacity perspective, the study of intersection is very important for the traffic engineers.

Interchange

Interchange is a system where traffic between two or more roadways ows 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

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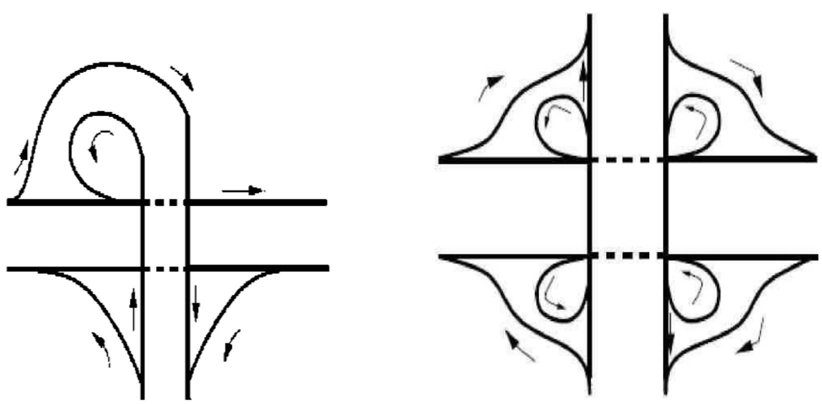
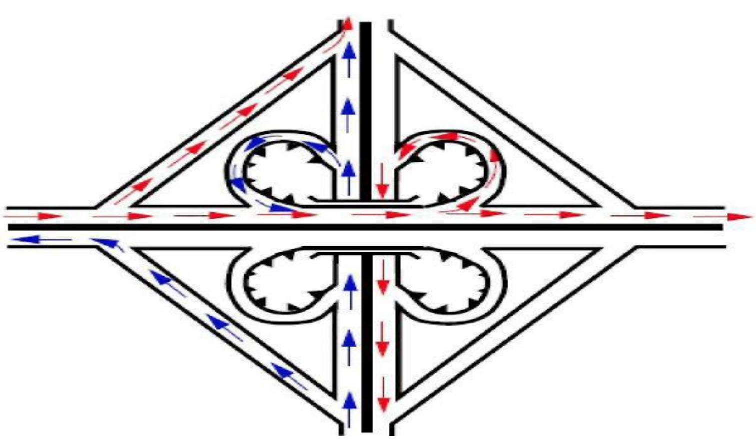
Channelized intersection

The vehicles approaching the intersection are directed to definite paths by islands, marking etc. and this process is called channelization. There can be channelized or unchannalized intersection but channelized intersection provides more safety and efficiency. It reduces the number of possible conflicts by reducing the area of conflicts available in the carriageway. If no channelizing is provided, the driver will have fewer tendencies to reduce the speed while entering the intersection from the carriageway. The presence of traffic islands, markings etc. makes the driver to reduce the speed and becomes more cautious while maneuvering the intersection. A channelizing island serves as a refuge for pedestrians and makes pedestrian crossing safer. Channelization of traffic through a three-legged intersection and a four-legged intersection is shown in the figure.



Clover-leaf intersection



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.

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UNIT 7

TRAFFIC ROTARIES

Rotary intersections or round abouts are special form of at-grade intersections laid out for the movement of traffic in one direction round 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

Advantages and disadvantages of rotary

The key advantages of the rotary intersection are listed below:

- Traffic flow is regulated to only one direction of movement, thus eliminating severe conflicts between crossing movements.
- All the vehicles entering the rotary are gently forced to reduce the speed and continue to move at slower speed. Thus, more of the vehicles need to be stopped.
- Because of lower speed of negotiation and elimination of severe conflicts, accidents and their severity are much less in rotaries.
- Rotaries are self-governing and do not need practically any control by police or traffic signals.
- They are ideally suited for moderate traffic, especially with irregular geometry, or intersections with more than three or four approaches.

Although rotaries offer some distinct advantages, there are few specific limitations for rotaries which are listed below.

- All the vehicles are forced to slow down and negotiate the intersection. Therefore the cumulative delay will be much higher than channelized intersection.
- Even when there is relatively low traffic, the vehicles are forced to reduce their speed.
- Rotaries require large area of relatively at land making them costly at urban areas.
- Since the vehicles are not stopping, and the vehicles accelerate at rotary exits, they are not suitable when there are high pedestrian movements

Traffic operations in a rotary

The traffic operations at a rotary are three; diverging, merging and weaving. All the other Conflicts are converted into these three less severe conflicts.

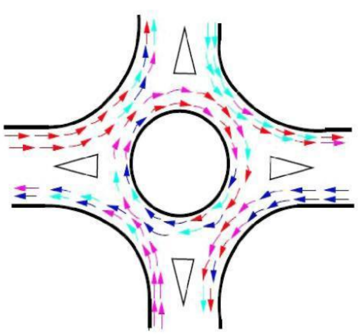
- **Diverging**: It is a traffic operation when the vehicles moving in one direction is separated into different streams according to their destinations.
- **Merging**: Merging is the opposite of diverging, when traffic streams coming from various places and going to a common destination are joined together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

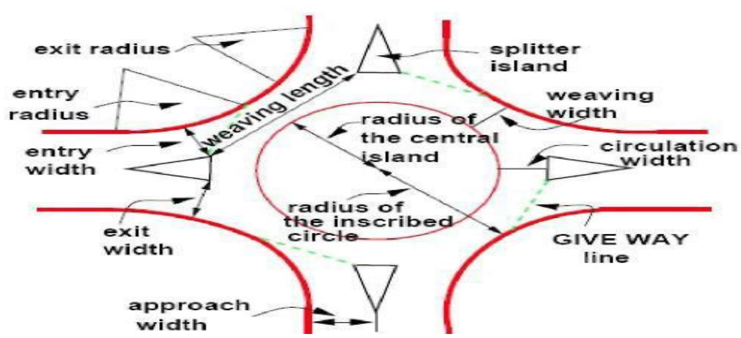
Design elements

The design elements include design speed, radius at entry, exit and the central island, weaving length and width, entry and exit widths. In addition the capacity of the rotary can also be determined by using some empirical formulae. A typical intersection is shown in figure

Design speed

All the vehicles are required to reduce their speed at a rotary. Therefore, the design speed of a rotary will be much lower than the roads leading to it. Although it is possible to design roundabout without much speed reduction, the geometry may lead to large size incurring huge cost of construction. The normal practice is to keep the design speed as 30 and 40 kmph for urban and rural areas respectively





Entry, exit and island radius

The radius at the entry depends on various factors like design speed, super elevation, and coefficient of friction. The entry to the rotary is not straight, but a small curvature is introduced. This will force the driver to reduce the speed. The speed range of about 20 kmph and 25 kmph is ideal for a radius and the radius of the rotary island so that the vehicles will discharge from the rotary at a higher rate. A general practice is to keep the exit radius as 1.5 to 2 times the entry radius. However, if pedestrian movement is higher at the exit approach, then the exit radius could be set as same as that of the entry radius. The radius of the central island is governed by the design speed, and the radius of the entry curve. The radius of the central island, in practice, is given a slightly higher reading so that the movement of the traffic already in the rotary will have priority of movement. The radius of the central island which is about 1.3 times that of the entry curve is adequate for all practical purposes.

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 7m width should be kept as 7m for urban roads and 6.5m for rural roads. Further, a three lane road of 10.5m is to be reduced to 7 and 7.5m 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

$$, W\ weaving = \frac{(e1 + e2)}{2} \quad 3.5m$$

Where e_1 is the width of the carriageway at the entry and e2 is the 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-weaving 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

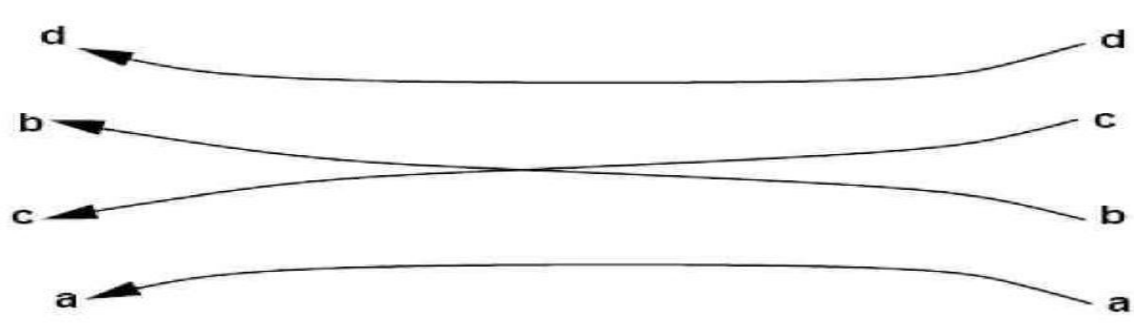
Capacity

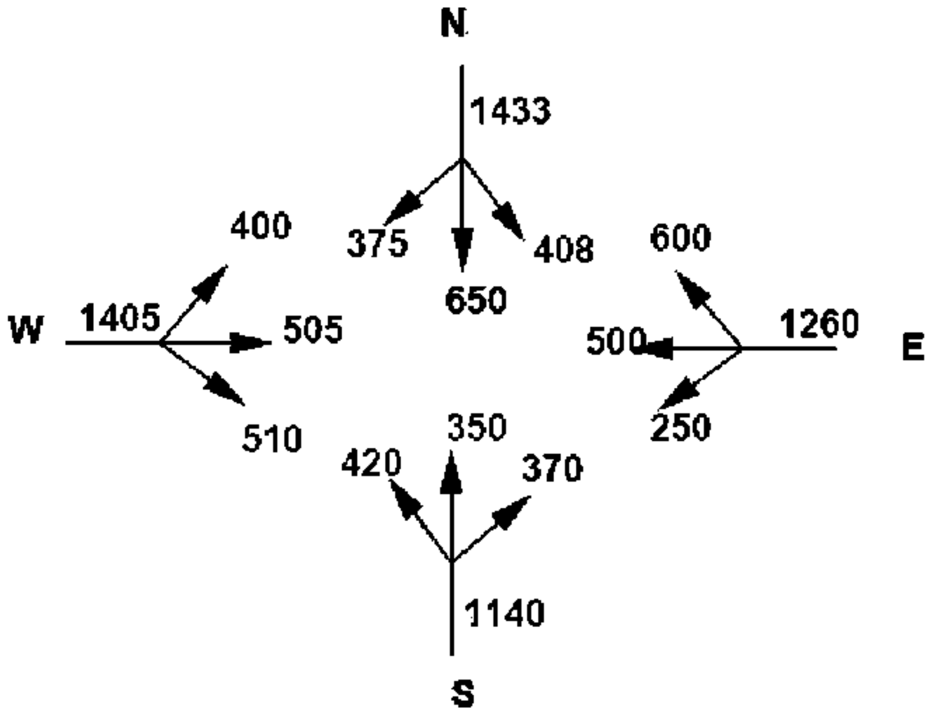
The capacity of rotary is determined by the capacity of each weaving section. Transportation road research lab (TRL) proposed the following empirical formula to find the capacity of the weaving section

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

Where e is the average entry and exit width, i.e., $(e_1+e_2)/2$, w is the weaving width, l is the length of weaving, and p is the proportion of weaving traffic to the non-weaving traffic. Shows four types of movements at a weaving section, a and d are the non-weaving traffic and b and c are the weaving traffic. Therefore

$$p = \frac{b + c}{a + b + c + d}$$





- Weaving width at the rotary is in between 6 and 18 meters.
- The ratio of average width of the carriage way at entry and exit to the weaving width is in the range of 0.4 to 1.
- The ratio of weaving width to weaving length of the roundabout is in between 0.12 and 0.4.
- The proportion of weaving traffic to non-weaving traffic in the rotary is in the range of 0.4 &1.
- The weaving length available at the intersection is in between 18 and 90

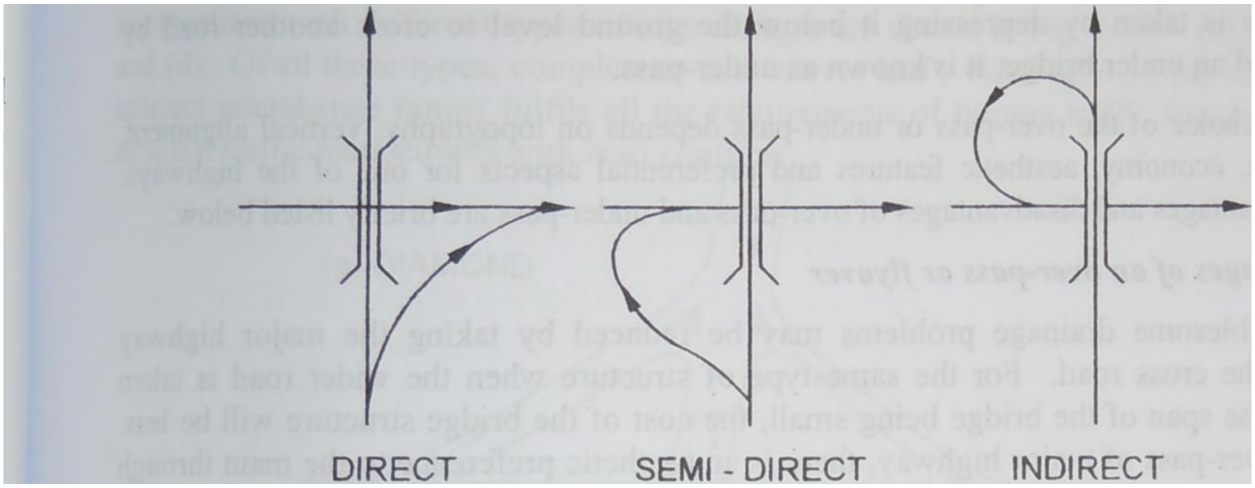
Grade Separated Intersections

Functioning of grade separated intersection and interchange ramps

Grade separated intersection design is the highest form of intersection treatment This type of intersection causes least delay and hazard to the crossing traffic and in general is much superior to intersections at-grade from the point of view of traffic safety, operation and capacity.

A highway grade separation is achieved by means of vertical level separation intersecting roads by means of a bridge thus eliminating all crossing conflicts at the intersection. The grade separation may be either by an over bridge/flyover or under pass. Transfer of route at the grade separation or the turning facilities are provided by 'interchange facilities' consisting of interchange ramps. Interchange ramps may be classified as: (i) direct, (ii) semi-direct and (iii) indirect ramps as shown in Fig below.

The direct interchange ramp involves diverging to right side and merging from the right; both these man oeuvres involve conflict with through traffic and therefore this type of interchange ramp is not free from the conflicts. Semi-direct interchange ramp allows diverging to left but merging is from right side; thus only the merging manoeuvre from the right causes conflict with through traffic. In the indirect interchange ramp, a simple diverging to the left and a merging from the left side are involved; thus both these manoeuvres are simpler, least hazardous and are free from major conflicts; but the distance to be traversed in indirect interchange is more.



Advantages of grade separated intersections

(a) Uninterrupted flow is possible for the crossing traffic. As the roads are separated at two levels, there is no crossing conflict thus avoiding accidents while crossing with no need to stop while crossing

(b) There is increased safety for turning traffic and by indirect interchange ramp even right turn movement is made quite easy and safe by converting into diverging to left and merging from left

(c) There is overall increase in comfort and convenience to the motorists and saving in travel time and vehicle operation cost
(d) The capacity of the grade separated intersection can practically approach the total capacity of the two cross roads
(e) Grade separation is an essential part of controlled access highway like expressway and freeway
(f) It is possible to adopt grade separation for all likely angles and layout of intersecting roads
(g) Stage construction of additional ramps is possible after the grade separation structure between main roads is constructed.

Disadvantages of grade separation

(a) It is very costly to provide complete grade separation and interchange facilities 1

(b) Where there is limited right of way like built up or urban area or where the, topography is not favorable, construction of grade separation is costly difficult and undesirable.

(c) In flat or plain terrain, grade separation may introduce undesirable crests and ags in the vertical alignment.

Types of grade separation structures

The grade separation intersection structures are classified as:

(i) Over-pass 0r flyover and

(ii) Under-pass.

When the major highway is taken above by raising profile above the general ground level by embankment or viaduct and an over-bridge across another highway, it is called an over-pass or flyover. On the contrary if ^ highway is taken by depressing it below-the ground level to cross another road by means of an under bridge, it is known as under-pass. The choice of the over-pass or under-pass depends on topography, vertical alignment , drainage, economy, aesthetic features and preferential aspects for one of the highways. The advantages and disadvantages of over-pass and under-pass are briefly listed below.

Advantages of an over-pass or flyover

Troublesome drainage problems may be reduced by taking the major highway ; above the cross road. For the same type of structure when the wider road is taken above the span of the bridge being small, the cost of the bridge structure will be less,

In an over-pass of major highway, there is an aesthetic preference to the main through traffic and less feeling of restriction or confinement when compared with the under-pass. Future expansion or lateral expansion or construction of separate bridge structure for divided highway is possible^

Disadvantages of an over-pass

In rolling terrain, if the major road is to be taken above, the vertical profile will also have undulating grade line. If the major highway is to be taken over by constructing high embankments or viaduct and by providing steep gradients, the increased grade resistance may cause speed reduction on heavy vehicles. Also, there will be restrictions to sight distance unless long vertical curves are provided.

Advantages of an under-pass

The presence of an under pass which can be seen from distance and so, there is a warning to traffic in advance. When the major highway is taken below, it is advantageous to the turning traffic because the traffic from the cross road can accelerate while descending the ramp to the major highway and the traffic from the major highway can decelerate while ascending the ramp to the cross roads. The under-pass may be of advantage when the main highway is taken along the existing grade without alteration of its vertical alignment and cross road is depressed and taken underneath.

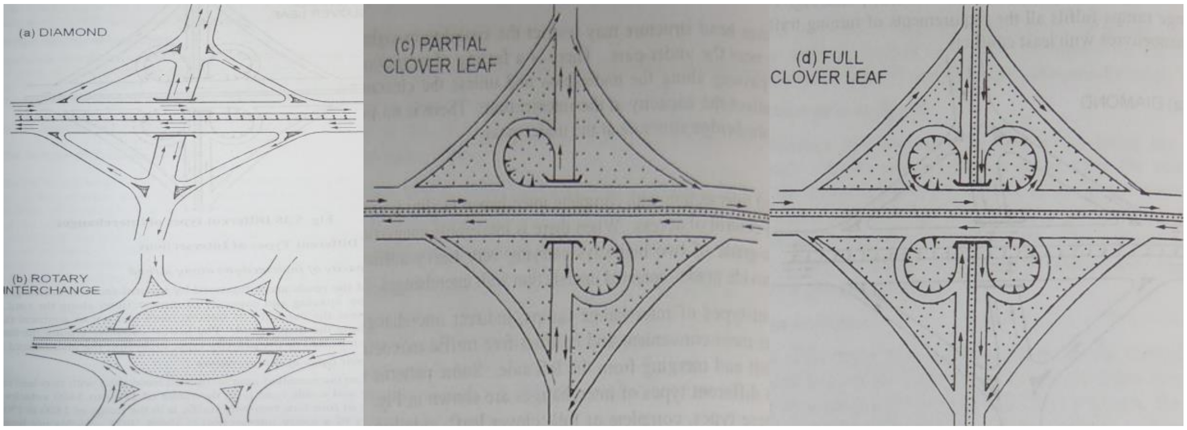
Disadvantages of an under-pass

There may be troublesome drainage problems at the under pass, especially when the ground water level rises high during rainy season and the road at the under-pass is to be depressed as much as 5 to7 m below the ground level. It may be necessary even to pump water continuously during the period when water-logging problems exist .At under-pass the overhead structure may restrict the visibility or sight distance even at valley curve near the under-pass. There is a feeling of restriction to the traffic at 3 sides while passing along the under-pass and unless the clearance is sufficiently large, this may affect the capacity at the intersection. There is no possibility of stage construction for the bridge structure at the under-pass.

Interchanges

Grade separated intersection with complete interchange facility is essential to develop a highway with full control of access. When there is intolerable congestion and accidents at the intersection at-grade of two highways carrying very heavy traffic there is no better Ration than to provide grade separated intersection with interchanges.

Of the different types of interchange ramps, indirect interchange ramp (vide Fig.) provides the most convenient and conflict-free traffic manoeuvres consisting of diverging to the left and merging from the left side. Some patterns of grade separated intersections with different types of interchanges are shown in Fig. Of all these types, complete or full ‘clover leaf, with four indirect interchange ramps fulfills all the requirements of turning traffic involving the simplest traffic manoeuvres with least conflicts.



UNIT 8

HIGHWAY DRAINAGE INTRODUCTION

Highway drainage is the process of removing and controlling excess surface and sub- soil water within the right of way this includes interception and diversion of water from the road surface and subgrade. The installation of suitable surface and sub-surface drainage system is an essential part of highway design and construction. During rains, part of the rain water flows on surface and part of it percolates through the soil mass as gravitational water until it reaches the ground water below the water table. Removal and diversion of surface water from the roadway and adjoining land is termed as surface drainage. Diversion or removal of excess soil-water from the subgrade is termed as sub-surface drainage. Some water is retained in the pores of the soil mass and on the surface of soil particles by surface tension and adsorptive forces, which cannot be drained off by normal gravitational methods and this water is termed as held water.

IMPORTANCE OF HIGHWAY DRAINAGE

Significance of Drainage

An increase in moisture content causes decrease in strength or stability of a soil mass the variation in soil strength with moisture content also depends on the soil type and the mode of stress application.

Highway drainage is important because of the following reasons:-

- Excess moisture in soil subgrade causes considerable lowering of its stability the pavement is likely to fail due to subgrade failure.
- Increase in moisture cause reduction in strength of many pavement materials like stabilized soil and water bound macadam.
- In some clayey soils variation in moisture content causes considerable variation in flume of subgrade. This sometimes contributes to pavement failure.
- One of the most important causes of pavement failure by the formation of waves and corrugations in flexible pavements is due to poor drainage.

- Sustained contact of water with bituminous pavements causes failures due to stripping of bitumen from aggregates like loosening or detachment of some of the bituminous pavement layers and formation of pot holes.
- TIE- prime cause of failures in rigid pavements by mud pumping is due to the presence of water in fine subgrade soil.
- Excess water on shoulders and pavement edge causes considerable damage.
- Excess moisture causes increase in weight and thus increase in stress and simultaneous reduction in strength of the soil mass. This is one of the main reasons of failure of earth slopes and embankment foundations.
- In places where freezing temperatures are prevalent in winter, the presence of water in the subgrade and a continuous supply of water from the ground water can cause considerable damage to the pavement due in frost action.
- Erosion of soil from top of unsurfaced roads and slopes of embankment, cut and hill side is also due to surface water.

Requirements of Highway Drainage System

- The surface water from the carriageway and shoulder should effectively be drained off without allowing it to percolate to subgrade.
- The surface water from the adjoining land should be prevented from entering the roadway.
- The side drain should have sufficient capacity and longitudinal slope to carry away all the surface water collected.
- Flow of surface water across the road and shoulders and along slopes should not cause formation of cress ruts or erosion.
- Seepage and other sources of underground water should be drained off by the subsurface drainage system.
- Highest level of ground water table should be kept well below the level of subgrade, preferably by at least 1.2 m.
- In waterlogged areas special precautions should be taken, especially if detrimental salts are present or if flooding is likely to occur.

SURFACE DRAINAGE SYSTEM FOR ROADS

Components of Surface Drainage System

The surface water from the roadway and the adjoining land is to be collected and then disposed off with the help of surface drainage system. The various components of the surface drainage system are: (a) the cross slope or camber of the pavement surface and the shoulders, (b) the road side drains and (c) cross drains.

This surface water is collected in longitudinal side drains and then the water is disposed off at the nearest stream, valley or water course with the help of cross drains and cross drainage structures. The cross section and slope of the road side drains are designed based on the estimated flow of surface water. Cross drainage structures like culverts and small bridges may be necessary for the disposal of surface water from the road side drains including the water of the streams crossing the roadway.

Collection of Surface Water

- **Camber or cross slope**

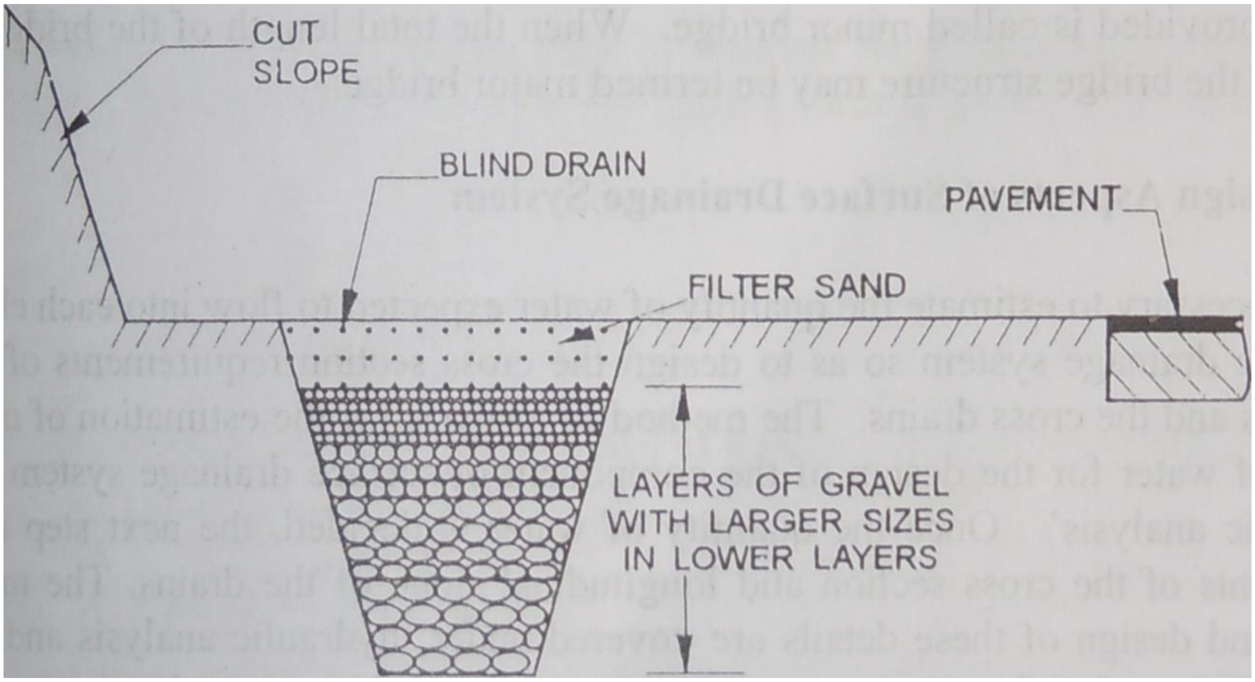
The water from the pavement surface and shoulders is first drained off to the road side drains with the help of the cross slope or camber. The rate of this cross slope of the pavement or the carriageway is decided based on: (i) the type of pavement surface and (ii) amount rainfall in the region. The values of camber range from 1 in 25 or 4.0 % for earth roads to 1 in 60 or 1.7 % for high type bituminous surface and CC pavements.

The recommended camber or cross slope for the earth shoulders ranges from 4.0 to 5.0 % depending on the soil type and rain fall in the region. In rural highways, the water which is drained from the pavement surface drains across the shoulders and is lead to the road side drains.

- **Road side drains**

The road side drains of highways passing through rural areas are generally open, unlined or 'kutcha' drains of trapezoidal shape, cut to suitable cross section and longitudinal. These side drains are provided parallel to the road alignment and hence these are also called longitudinal drains. On plain terrain with embankments the longitudinal drains are provided on both sides beyond the toe of the embankment. However if the road passes through sloping terrain (with cross slope more than 4.0 %) the longitudinal drain may be provided on one side only beyond the toe of the embankment along the higher side of the slope. The water on the lower side of the road will continue to drain away towards the natural valleys.

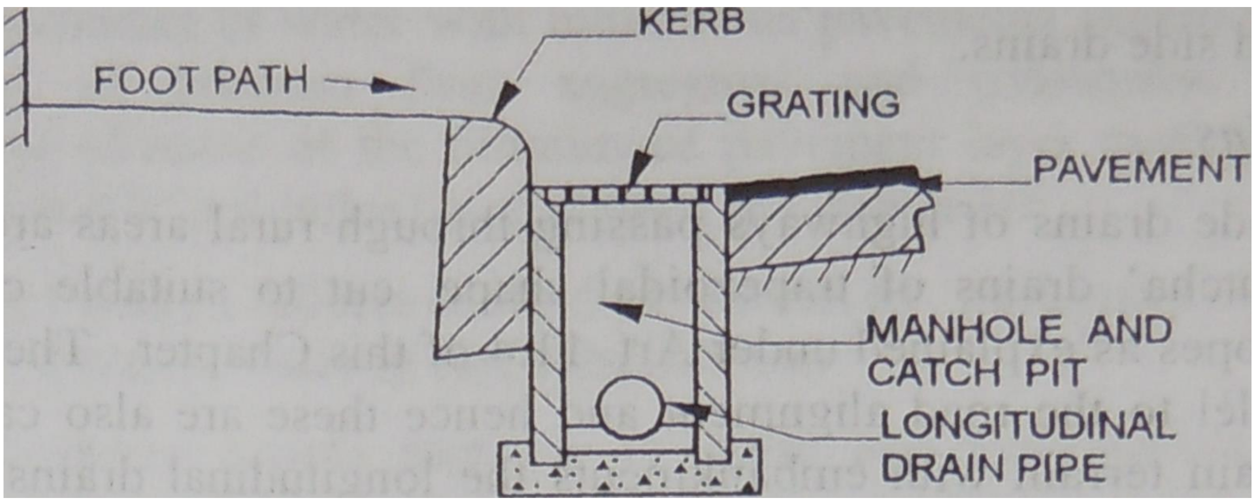
In cuttings, the longitudinal drains are installed on either side of the formation. But in places where there is restriction of space, construction of deep open drains may be undesirable. This is particularly true when the road formation is in cutting. In such cases drainage trenches of suitable depth and cross section are dug and properly filled with layers of filter material consisting of coarse sand and gravel to form the 'covered drain' as shown in figure below.



Drainage trench filled with filter material

On urban roads because of the limitation of land width and also due to the presence of foot path, dividing islands and other road facilities, it is necessary to provide underground longitudinal drains. Water drained from the pavement surface can be carried forward along the under-ground longitudinal drains installed between the kerb and the pavement for short distances. This water may be collected in catch pits at suitable intervals and lead through under-ground drainage pipes. Section of a typical catch pit with grating to prevent the entry of rubbish into the drainage system of urban roads is shown in figure below

Drainage of surface water is all the more important on hill roads. Apart from the drainage of water from the road formation, the efficient diversion and disposal of water flowing down the hill slope across the road and that from numerous cross streams is an important part of hill road drainage system. If the drainage system on hill road is not adequate and efficient, it will result in complex maintenance problems.



Surface drainage system on urban roads

- **Cross drains**

On rural stretches of highways, the water flowing along the road side drains are collected by suitable cross drains through cross drainage structures (CD structures) at locations of natural valleys and streams and disposed off to the natural water course. These CD structures may generally be a suitable type of culvert, depending on the quantity of water to be carried across and the span. Different types of culverts adopted on rural stretches of highways are, slab, box or pipe culverts. The CD structures should extend up to the full formation width (including the roadway and the shoulders) and the ends may be protected by suitable abutments and parapet walls. When the width of the stream or river to be crossed is large (generally more than 6.0 m) the CD structure provided is called minor bridge. When the total length of the bridge is more than 60 m the bridge structure may be termed major bridge.

- **Design Aspects of Surface Drainage System**

It is necessary to estimate the quantity of water expected to flow into each element of the surface drainage system so as to design the cross section requirements of the road side drains and the cross drains. The method of analysis for the estimation of maximum quantity of water for the design of the component of surface drainage system is called 'hydrologic analysis'. Once the quantity of water is decided, the next step is design requirements of the cross section and longitudinal slope of the drains. The method of analysis and design of these details are covered under 'hydraulic analysis and design'. Thus the design of surface drainage system may be divided into two phases:

(i) Hydrologic analysis

(ii) (ii) Hydraulic analysis

HYDROLOGIC ANALYSIS

- **Objective and Principle**

The main objective of hydrologic analysis is to estimate the maximum quantity of water, Q expected to reach the component of the drainage system under consideration, such as the road side drain. A portion of the precipitation during the rain-fall in filtrates into the ground as ground water and a small portion get evaporated. The remaining portion of water which flows over the surface is termed as 'run-off. Various factors affecting the run-off are:

(a)Intensity or rate of rainfall

(b) Type of soil

(c) Moisture content in the soil

(d) Topography of the area and

(e) Type of ground cover like the type of pavement surface, vegetation on the adjoining land etc.

The surface drainage system is to be designed to drain away the surface run-off water reaching each component, such as the roadside drains and the cross drains. The following four steps are to be followed:

- to collect the details of rain fall in the area including intensity, duration and frequency of occurrence of storm
- to find drainage area from where water is likely to flow in
- to determine the run-off and maximum rate of run-off for the area under consideration using any of the accepted approaches
- to estimate the peak quantity of run-off water reaching the component of the drainage system to be designed

Estimation Quantity of Run-off Flowing into the Drain

'Rational Formula' is widely used to estimate the peak run-off water for highway drainage. The Rational formula, in its simplest form is given by the equation:

$$Q = C\ i\ A_d \qquad \text{......} (1)$$

Where

- Q = run-off, m^3/sec
- C = run-off coefficient, expressed as a ratio of run-off to rate of rain fall
- i = intensity of rain fall, mm/sec
- A_d=drainage area in 1000 m

It may be noted that the above expression is dimensionally not balanced. As per Rational Formula, the quantity of run-off flowing into the drain depends upon the run-off coefficient, intensity of rain fall and the total drainage area.

- **Run-off coefficient, C**

The value of run-off coefficient, C depends mainly on the type of surface and its slope. The C-values may be taken as, (a) 0.8 to 0!9 for bituminous and cement concrete pavements, (b) 0.35 to 0.70 for gravel and WBM pavements, (c) 0.40 to 0.65 for impervious soil, (d) 0.30 to 0.55 for soil covered with turf and (e) 0.05 to 0.30 for previous soils.

- **Drainage Area, A_d**

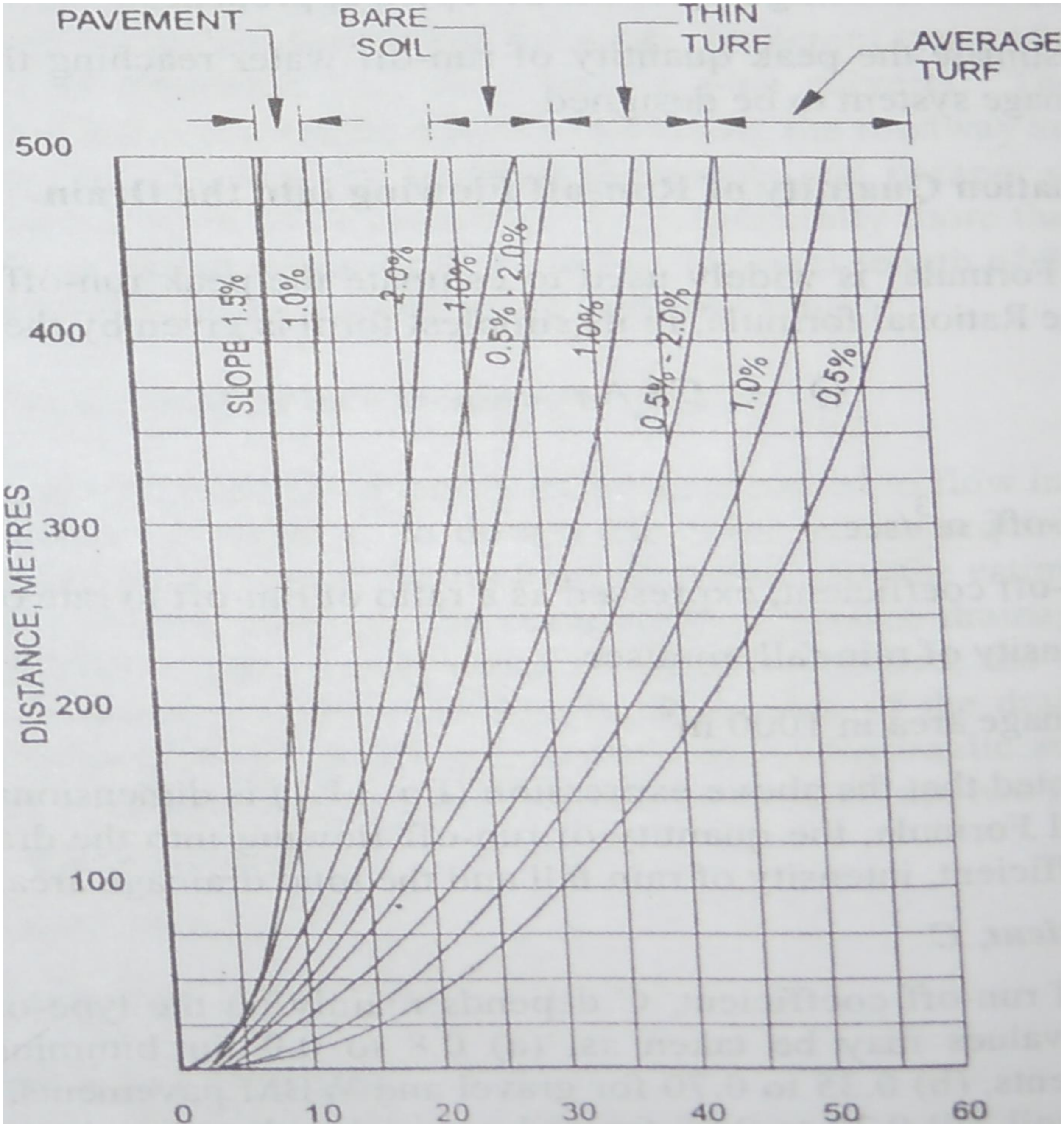
The drainage area from which the surface water is expected to flow to a side drain is determined with the aid of a contour map or by studying the topography of drainage area. This area is expressed in units of 1000 square metre to obtain the value of A_d to be used in Eq. 1

When the drainage area, A_d consists of several types of surfaces with different values of run-off coefficients C_1 C_2, C_3 ...and if their respective areas are A_1, A_2, A_3,...., the weighted average value of run-off coefficient, C is determined from the equation:

$$C = \frac{A\mathbf{1}C1 + A2C2 + A3C3}{A1 + A2 + A3}$$

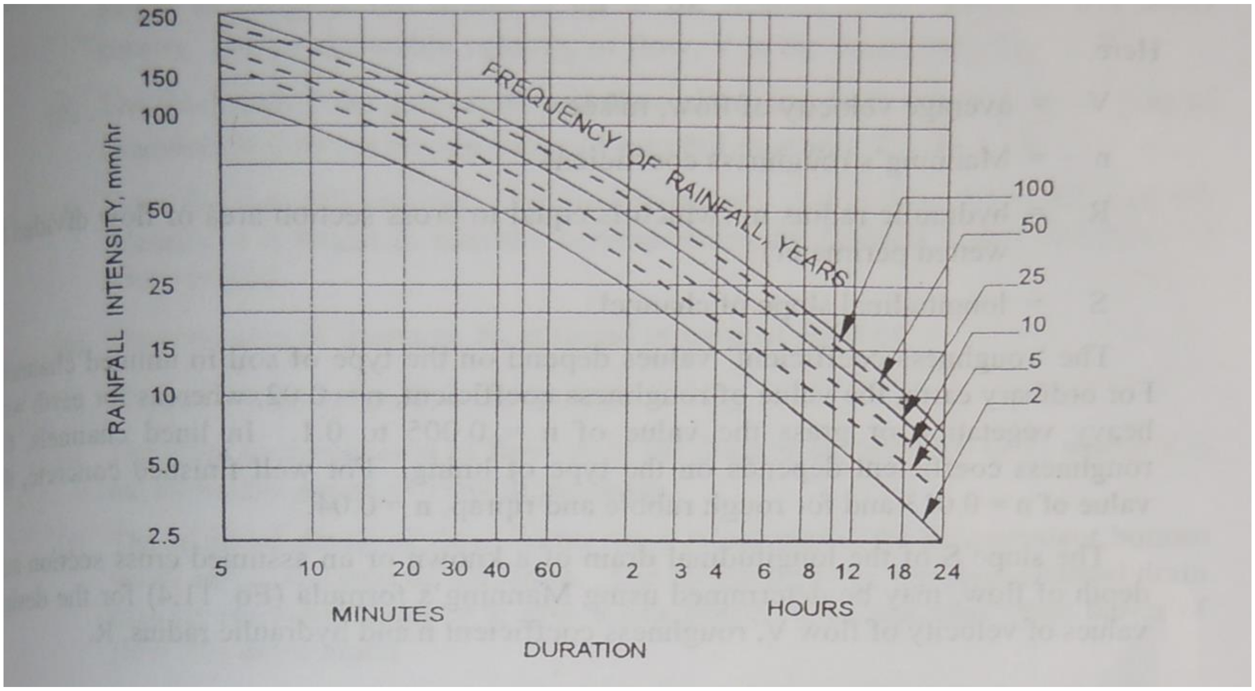
- **Design value of rainfall intensity, i**

The design value of the rain fall intensity is to be determined for the expected duration of storm and frequency of occurrence. Therefore the inlet time for the storm water to flow from the remotest point in drainage area to the drain inlet is estimated using the Chart The time for water to flow through the drain between the inlet and outlet points is determined based on the allowable velocity of flow in the drain, which generally ranges from 0.3 to 1.5 m/sec, depending on soil type. In the case of surfaced drains and pipe drains higher velocity of flow is permitted.



Time of flow to inlet

The time of concentration or the duration of storm for design may be taken as the sum of inlet time and the time of flow through the drain. The frequency of occurrence storm or the 'return period' may be taken as 5, 10, 25 or 50 years. From the chart below the design value of the rainfall intensity, i is found corresponding to duration of storm and the selected value of frequency.



Typical rainfall intensity — duration curves

- **Design value of run-off, Q**

Thus with the estimated value of C, i and A<j the design value of run-off, Q for the longitudinal side drain is determined: Q = C i Ad (vide Eq. 1)

HYDRAULIC DESIGN

Hydraulic Design Elements of Drains

Once the design runoff Q is determined, the next step is the hydraulic design of drains. The side drains and partially filled culverts are designed based on the principles of flow through open channels.

If Q is the quantity of surface water (m^3/sec) to be removed by a side drain and V is the allowable velocity of flow (m/sec) on the side drain, the area of cross section A of the channel (m^3) is found from the relation:

$$Q = AV \quad \text{(Eq. 2)}$$

The velocity of unlined channel must be high enough to prevent silting and it should not be too high as to cause erosion. The allowable velocity of flow depends on the soil type of the open side drain. The desirable values of velocity of flow are, 0.3 to 0.5 m/sec for sand and silt, 0.6 to 0.9 for loam, 0.9 to 1.5 for clay, 1.2 to 1.5 for gravel and 1.5 to 1.8 m/sec for good soil covered with well-established grass.

The velocity of flow of water along the drain depends on its longitudinal slope. Assuming uniform and steady flow through channel of uniform cross section and slope, 'Manning's formula' is used for determining the velocity of flow or the longitudinal slope which is given by:

$$V = \frac{1}{\chi} * R^{\left(\frac{2}{3}\right)} * S^{\frac{1}{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(Eq. 3)}$$

Here,

- V = average velocity of flow, m/sec
- n = Manning's roughness coefficient.
- R = hydraulic radius m (which is equal to cross section area of flow divided by wetted perimeter)
- S = longitudinal slope of channel.

The 'roughness coefficient' values depend on the type of soil in unlined channels. For ordinary earth, the value of roughness coefficient, n = 0.02, whereas for earth with heavy vegetation or grass the value of n = 0.005 to 0.1. In lined channels, the roughness coefficient depends on the type of lining. For well finished concrete, the value of n = 0.013 and for rough rubble and riprap, n = 0.04.

The slope S of the longitudinal drain of a known or an assumed cross section and depth of flow, may be determined using Manning's formula (Eq. 3) for the design values of velocity of flow V, roughness coefficient n and hydraulic radius, R.

- **Hydraulic Design Data**

The following data are to be collected for the design of road side drain:

- Total road length and width of land from where water is expected to flow on the stretch of the side drain
- Run-off coefficients of different types of surfaces in the drainage area and their respective areas (such as paved area, road shoulder area, turf surface, etc.)
- Distance from farthest point in the drainage area to the inlet of the side drain along the steepest gradient and the average value of the slope
- Type of soil of the side drain, the value of roughness coefficient and the allowable velocity of flow in the drain
- Rain fall data including average intensity and frequency of recurrence of flood

Design Steps

Simplified steps for the design of longitudinal road side drain are given below:

1. The frequency of return period (such as 10 years, 25 years etc.) is decided based on finances available and desired margin of safety, for the design of the drainage system
2. The values of coefficients of run-off C_1, C_2, C_3 etc. from drainage areas A_1, A_2, A_3 etc. are found and the weighted value of C is computed
3. Inlet time, Ti for the flow of storm water from the farthest point in the drainage area to the drain inlet along the steepest path of flow is estimated from the distance, slope of the ground and type of the cover.
4. Time of flow along the longitudinal drain, T_2 is determined for the estimated length of longitudinal drain, L up to the nearest cross drainage or a water course. For an allowable velocity of flow, V in the drain, time $T_2 = L/V$
5. The total time T for inlet flow and flow along the drain is taken as the time of concentration or the design value of rain fall duration, $T = T_1 + T_2$
6. From the rain fall intensity duration frequency curves the rain fall intensity, i is found in mm/sec corresponding to duration T and frequency of return period
7. The total area of drainage A_d is found in units of 1000 m^2
8. The run-off quantity, Q is computed = $C\ i\ A_d$

9. The cross sectional area of flow, A of the drain is calculated = Q/V, where V is the allowable speed of flow in the drain
10. The required depth of flow in the drain is calculated for a convenient bottom width and side slope of the drain. The actual depth of the open channel drain may be increased slightly to give a free board. The hydraulic mean radius of flow R is determined
11. The required longitudinal slope, S of the drain is calculated using Manning's formula adopting suitable value of roughness coefficient, n.