



ANNAMACHARYA INSTITUTE OF TECHNOLOGY & SCIENCES

(AUTONOMOUS)

UTUKUR (P), C. K. DINNE (V&M), KADAPA, YSR DIST.

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Highway Engineering

(23HPC0118)

Lecture Notes

By

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CIVIL ENGINEERING						
III B.Tech – II Semester						
Course Code	HIGHWAY				L	T
23HPC0118	ENGINEERING				3	0
					P	C
					0	3

Course Objectives:

The objectives of this course are to make the student to:

1. **Understand** the history, importance, and planning aspects of highway development in India.
2. **Apply** geometric design principles for highway alignment, sight distance, and curves.
3. **Analyze** traffic characteristics, capacity, level of service, and road safety measures.
4. **Design** flexible and rigid pavements using IRC guidelines.
5. **Evaluate** highway construction materials, testing methods, and maintenance techniques.

Course Outcomes:

Upon successful completion of this course, students will be able to:

1. **Explain** the significance, planning, and alignment of highways
2. **Design** geometric elements of highways, including curves, gradients, and sight distances.
3. **Analyze** traffic flow, capacity, level of service, and implement road safety measures.
4. **Design** flexible and rigid pavements as per IRC guidelines.
5. **Assess** construction practices, highway materials, and pavement maintenance techniques.

CO – PO Articulation Matrix:-														
Course Outcomes	PO 1	PO 2	PO 3	PO 4	PO 5	PO 6	PO 7	PO 8	PO 9	PO 10	PO 11	PO 12	PS O1	PS O2
CO -1	3	2	2	1	-	-	-	-	-	1	-	2	3	2
CO -2	3	3	3	2	2	-	-	-	-	-	-	1	3	3
CO -3	3	3	3	2	2	-	2	1	-	-	-	1	3	3
CO -4	3	3	3	2	2	-	2	1	-	-	-	1	3	3
CO -5	2	2	2	1	3	-	3	1	-	1	1	2	2	2

UNIT – I

PLANNED HIGHWAY DEVELOPMENT in INDIA

Highway development in India – Necessity for Highway Planning- Different Road Development Plans- Classification of Roads- Road Network Patterns – Highway Alignment- Factors affecting Alignment- Engineering Surveys – Drawings and Reports.

UNIT – II

GEOMETRIC DESIGN of HIGHWAYS

Importance of Geometric Design- Design controls and Criteria- Highway Cross Section Elements- Sight Distance Elements- Stopping sight Distance, Overtaking Sight Distance and intermediate Sight Distance- Design of Horizontal Alignment- Design of Super elevation and Extra widening- Design of Transition Curves-Design of Vertical alignment-Gradients- Vertical curves.

UNIT – III

TRAFFIC ENGINEERING STUDIES

Basic Parameters of Traffic-Volume, Speed and Density – Definitions and their inter relation – Highway capacity and level of service concept – factors affecting capacity and level of service - Traffic Volume Studies- Data Collection and Presentation-Speed studies- Data Collection and Presentation- - Road Accidents-Causes and Preventive measures- Accident Data Recording – Condition Diagram and Collision Diagrams.

UNIT – IV INTERSECTION DESIGN

Conflicts at Intersections- Channelization: Objectives –Traffic Islands and Design criteria- Types of At-Grade Intersections – Types of Grade-Separated Intersections- Rotary Intersection – Concept of Rotary and Design Criteria- Advantages and Disadvantages of Rotary Intersections.

UNIT – V PAVEMENT DESIGN

Types of Pavements – Difference Between Flexible and Rigid Pavements – Pavement Components – Sub Grade, Sub Base, Base and Wearing Course – Functions of Pavement Components – Design Factors – Flexible Pavement Design Methods – G.I Method, CBR Method, (As Per IRC 37-2015) –Design of Rigid Pavements (As Per IRC 58-2018) – Critical Load Positions – Westergaard's Stress Equations – Computing Radius of Relative Stiffness and Equivalent Radius of Resisting Section – Stresses in Rigid Pavements – Design of Expansion and Contraction Joints in CC Pavements. Design of Dowel Bars and Tie Bars.

TEXT BOOKS:

1. Highway Engineering – S.K.Khanna&C.E.G.Justo, Nemchand& Bros., 9th edition (2011).
2. Transportation Engineering, Volume I, C Venkatramaiah, Universities Press, 2015.

REFERENCE BOOKS:

1. Principles of Highway Engineering by L.R.Kadiyali, Khanna Publishers
2. Traffic Engineering and Transportation Planning by L.R.Kadiyali and Lal-Khanna Publications 9th edition
3. Highway Engineering – Dr. S.K.Sharma, S.Chand Publishers 2014 edition
4. IRC 37-2015 - Guidelines for Design of flexible pavements
5. IRC 58-2018 - Guidelines for Design of rigid pavements

NOTE: CODE BOOKS ARE ALLOWED TO EXAMINATION HALL

Online Learning Resources:

<https://nptel.ac.in/courses/105101087>

UNIT – I
PLANNED HIGHWAY DEVELOPMENT in INDIA

Importance of Transportation

Transportation is a toll to measure the Economic, Industrial, Social and Cultural development of any country. It is vital for the economic development of any region people and the communities are essentially to be transported material from one place to other. In the production stage transportation is required for carrying raw materials like seeds, manure, coal, steel etc. In the distribution stage transportation is required from the production centers via; farms and factories to the marketing centers and later to the retailers and the consumers for distribution. The importance of transportation may include:

- **Availability of raw materials:** Transportation helps in carrying the raw materials from one place to another place. Initially raw materials are made at one place and are being transported to another place for processing and for manufacturing goods.
- **Availability of goods to the customer:** The goods are being transported from one place to another place. These goods which are produced at one place are transported to other distant places for their usage. It flexibly moves the goods from one place to another place.
- **Enhances the standard of living:** It improves the standard of living. As the transportation of each and every good is being done then the productivity increases which results in the reduced or the effective costs. Because of reduction in the cost they can use different commodities for different purposes and can lead a secure life.
- **Helps a lot during the emergencies and even during natural disasters:** Transportation helps during the natural disturbances. It helps in quick moving from one place to another place and

supplies the required operations.

- **Helps for the employment:** Transportation provides employment for many people as drivers, captains, conductors, cabin crew and even the people are used for the construction of different types of transportation vehicles. And even by the use of transportation the remote people are being employed with the access to the urban facilities and the opportunities.
- **Helps in mobility of the laborers:** Many people are traveling to other countries on their employment basis. Transportation plays an important role in such cases.
- **Helps for bringing nations together:** Transportation on the whole is used for globalization i.e. it brings nations together and it creates awareness about the cultural activities and even about the industries and helps a lot for importing and exporting of different goods. These above are some of the necessities which make us to use transportation.

The importance and adequacy of transportation system of a country indicates its economic and social development.

Economic Activity: Two important factors well known in economic activity are:

1. Production or supply
2. Consumption for human wants or demand.

Economic activity is the process by means of which the products are utilized to satisfy human demand. The role of transportation in the economic activity starts its function from production stage to the final distribution. Increased productivity of various items such as agricultural and industrial products and their distribution through efficient transportation system can lower the cost of the products. The cost of transportation substantially influences the consumer price of the commodities.

Social Effects: The various social effects of transportation may be further classified into:

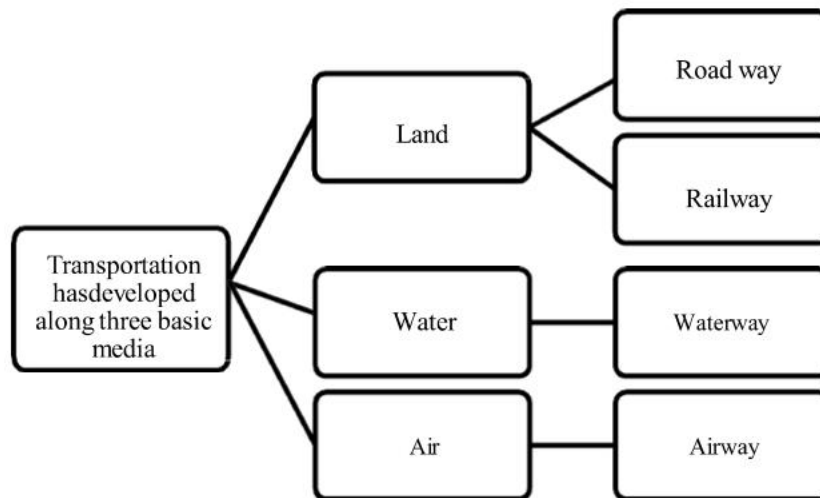
- **Sectionalism and transportation:** improved transportation has important implication in reducing sectionalism with in the country and also outside the country. More frequently travels in other part of the country and outside the country tends to increases knowledge of the people from other section of society.

Concentration of population into urban area: improves transportation facilities bring prosperity to the urban population. The employment opportunities, prosperity and superior facilities for education, medical care etc available in urban area attracts the population from other areas, resulting in enhanced economic activities. Inadequate transportation facilities lead to concentration of population in cities which often results in growth of slums in urban area, leading to various

associated problems. If efficient transportation facilities are available, the people would prefer to reside at localities away from urban centers and commute to the city for their work. In general, the transportation facilities are essential for the well-being of the community.

Aspect of safety, law and order: transport facilities are essential for restoring order to areas affected by an emergency to maintain law and order at home it is required to have an efficient system of transport network. To define the territory of the country against external aggression and to guard the borders with the foreign territories transport facilities are needed.

Different Modes of Transportation



Three basic modes of transport are by land, water and air. Land has given development of road and rail transport. Water and air have developed waterways and airways respectively. Apart from these major modes of transportation, other modes include pipelines, elevators, belt conveyors, cable cars, aerial ropeways and monorails. Pipe lines are used for the transportation of water, other fluids and even solid particles.

The four major modes of transportation are:

1. **Roadways or highways**
2. **Railways**
3. **Airways**
4. **Waterways**

Roadway

Advantages	Disadvantages
Lesser Risk of Damage in Transit	Seasonal Nature
Less Capital Outlay	Accidents and Breakdowns

Highway Engineering

Door to Door Service	Unsuitable for Long Distance and Bulky Traffic
Service in Rural Areas	Slow Speed
Flexible Service	Lack of Organization
Suitable for Short Distance	Large consumer of petroleum energy
Max service to one and all	
Private Owned Vehicles	
Feeder to other Modes of Transport	

Railway

Advantages	Disadvantages
Employment Opportunities	Huge Capital Outlay
Better Organized	Lack of Flexibility

High Speed over Long Distances	Lack of Door to Door Service
Suitable for Bulky and Heavy Goods	Unsuitable for Short Distance and Small Loads
Cheaper Transport	Booking Formalities
Larger Capacity	No Rural Service
Administrative Facilities of Government	Centralized Administration
Energy required is one fourth to one sixth of that required by the road	

Airway

Advantages	Disadvantages
Fastest mode of transport	The haul unit per unit length is costly.
Comfortable and Quick Services	Small Carrying Capacity
Emergency Services	Breakdowns and Accidents
Most Suitable for Carrying Light Goods of High Value	Large Investment
National Defense	Specialized Skill employees
Space Exploration	Unsuitable for Cheap and Bulky Goods
	Legal Restrictions

Waterway

Advantages	Disadvantages
Minimum energy to haul unit load through unit distance	Slowest mode of transport
Most energy efficient transport system	Operated only on sea routes between harbours and ports
Suitable for bulk cargo	Depends on weather condition
Leads to the development of the industries.	Ocean tides affects the loading and unloading operation
Cheapest: Cost per tonne is lowest	The route is circuitous.

Characteristics of Road Transport

- Roads are used by various types of road vehicles like cars, buses, truck, two and three wheeled automobiles, pedal cycles and animal drawn vehicles. But railway tracks are used only by rail locomotives and wagons, waterways are used only by ships and boats.
- Road transport requires a relatively small investment for the government. Motor vehicles are much cheaper than other carriers like rail locomotives and wagons, water and air carriers.
- Construction and maintenance of road is also cheaper than that of railway tracks, docks, harbors and airports.
- Road transport offers a complete freedom to road users to transfer the vehicle from one lane to another and from one road to another according to need and convenience. This flexibility of changes in location, direction, speed, and timing of travel is not available in other modes of transport.
- In particular, for short distances travel, road transport saves time.
- Speed of movement is directly related with the severity of accident. The road safety decreases with increasing dispersion in speed. Road transport is subjected to a high degree of accidents due to flexibility of movements offered to the road users.
- Road transport is the only means of transport that offers itself to the whole community alike.

Highway Development in India

In the beginning of fifth century A.D., emperor Ashoka had improved the roads and provided facilities for travelers. During the Pathan and Mughal periods, the roads of India were greatly improved during the time of Mughal period. Roads linking North-West and the Eastern areas through gangatic plains were built during this time. In 1865 Lord Dalhousie, governor general formed the public work department.

Jayakar Committee Recommendations and Implementation of Recommendations

Over a period after the First World War, motor vehicles using the roads increased and this demanded a better road network which can carry mixed traffic conditions. The existing roads were not capable of withstand the mixed traffic conditions. For the improvement of roads in India government of India appointed Mr. Jayakar Committee to study the situations and to recommend suitable measures for road improvement in 1927 and a report was submitted in 1928 with following recommendations:

- Road development in the country should be considered as a national interest. As the provincial and local government do not have the financial and technical capacity for road development.
- Extra tax to be levied from the road users as fund to develop road.

- A Semi-official technical body has to be formed to collect and pool technical knowhow from various parts of the country and to act as an advisory body on various aspects of the roads.
- A research organization should be instituted at National level to carry out research and development work and should be available for consultation.

Implementations

Majority of the recommendations were accepted by the government implemented by Jayakar Committee. Some of the technical bodies were formed such as,

- Central Road Fund (CRF) in 1929
- Indian Road Congress (IRC) in 1934
- Central Road Research Institute (CRRI) in 1950.

Central Research Fund (CRF)

- Central Research Fund (CRF) was formed on 1st March 1929.
- The consumers of petrol were charged an extra levy of 2.64 paisa/litre of petrol to build up this road development fund.

From the fund collected 20 percent of the annual revenue is to be retained as meeting expenses on the administration of the road fund, road experiments and research on road and bridge projects of special importance.

- The balance 80 percent of the fund to be allotted by the Central Government to the various states based on actual petrol consumption or revenue collected.
- The accounts of the CRF are maintained by the Accountant General of Central Revenues.
- The control of the expenditure is exercised by the Roads Wings of Ministry of Transport.
- At present the revised cess collected on petrol and high speed diesel towards CRF is @ Rs 2/ litre.

Indian Road Congress (IRC)

- It is a semi -official technical body formed in 1934.
- It was formed to recommend standard specifications.
- It was constituted to provide a forum of regular technical pooling of experience and ideas on all matters affecting the planning, construction and maintenance of roads in India.
- IRC has played an important role in the formulation of the 20-year road development plans in India.
- Now, it has become an active body of national importance controlling specifications, guidelines and other special publications on various aspects of Highway Engineering.

- The IRC publishes journals, research publications, standards, specifications, guidelines and other special publications on various aspects of highway engineering.

Central Road Research Institute (CRRI)

- CRRI was formed in the year 1950 at New Delhi
- It was formed for research in various aspect of highway engineering
- It is one of the National laboratories of the Council of Scientific and Industrial Research.
- This institute is mainly engaged in applied research and offers technical advice to state governments and the industry on various problems concerning roads.

Highway Research Board (HRB)

The HRB of the IRC was set up in 1973 with a view to give proper direction and guidance to road research activities in India. The objectives are to

- Ascertain the nature and extent of research required
- Correlate research information from various organizations in India and abroad with a view to exchange publications and information on roads.
- Co-ordinate and conduct correlation services
- Collect and disseminate results of research
- Channelize consultative services

Road Classification

Based on weather

- ❖ **All-weather roads:** These roads are negotiable during all weather, except at major river crossings where interruption of traffic is permissible up to a certain limit extent, the road pavement should be negotiable during all weathers.
- ❖ **Fair-weather roads:** On these roads the traffic may be interrupted during monsoon season at causeways where streams may overflow across the roads.

Based on the carriage way

- ❖ **Paved Roads:** These roads are provided with a hard pavement course which should be at least a water bound macadam (WBM) layer.
- ❖ **Unpaved Roads:** These roads are not provided with a hard pavement course of at least a WBM layer. Thus earth roads and gravel roads may be called as unpaved roads.

Based on Surface pavement provided

- ❖ **Surface Roads:** These roads are provided with a bituminous or cement concrete surfacing. Roads which are provided with bituminous surfacing are called as black topped roads and that of

concrete are referred to as concrete roads respectively.

- ❖ **Unsurfaced Roads:** These are not provided with bituminous or cement concrete surfacing.

Based on Traffic Volume

- ❖ Heavy traffic roads
- ❖ Medium traffic roads
- ❖ Light traffic roads

Based on Load transported or tonnage

- ❖ Class-I or Class-A
- ❖ Class-II or Class-B etc

Based on location and Function

The Nagpur road plan classified the roads in India based on location and function into following five categories

- ❖ **National Highways (NH):** The NH connects the capital cities of the states and the capital cities to the port. The roads connecting the neighbouring countries are also called as NH. The NH are at least 2 lanes of traffic about 7.5m d wide. The NH usually has concrete or bituminous surfacing. NH-1 is one of the national highways connecting Delhi to Amritsar.
- ❖ **State Highways (SH):** SH are the main roads within the state and connect important towns and cities of state. The width of state highways is generally 7.5m. The NH and SH have the same design speed and geometric design specifications. SH -88 is one of the state highway connecting Mysore – Hunsur- Periyapatna –Bylakuppe –Kushalanagar – Madkeri – Sulya - Puttur – Bhantwal.
- ❖ **Major District Roads (MDR):** These are the important roads within a district serving areas of production and markets and connecting with other major roads or main highways of a district. The MDR has lower speed and geometric design specification than NH/SH. The MDR should have at least metalled single lane carriage way (i.e., 3.8m) wide. The roads carry mixed traffic.
- ❖ **Other District Roads (ODR):** These roads serving rural areas of production and providing them outlet to market centres, taluk headquarters, block development headquarters or other main roads. These are of lower design specification than MDR. These roads have a single lane and carry mixed traffic.
- ❖ **Village Roads (VR):** These are roads connecting villages or groups of villages with each other to the nearest road of a higher category.

Modified Classification of Road system by Third Road Development Plan

- ❖ Primary System (Expressways and National Highways)
- ❖ Secondary System (State Highways and Major District Roads)
- ❖ Tertiary System (Other District Roads and Village Roads).

Expressways are a separate class of highways with superior facilities and design standards and are meant as through routes having very high volume of traffic. The expressways are to be provided with divided carriageways, controlled access, gradeseparations at cross roads and fencing. These highways should permit only fast moving vehicles. Expressway may be owned by the central government or state government, depending on whether the route is a NH or SH.

Road Patterns

1. Rectangular or block pattern

In this pattern, the whole area is divided into rectangular blocks of plots, with streets intersecting at right angles. The main road which passes through the center of the area should be sufficiently wide and other branch roads may be comparatively narrow. The main road is provided a direct approach to outside the city.

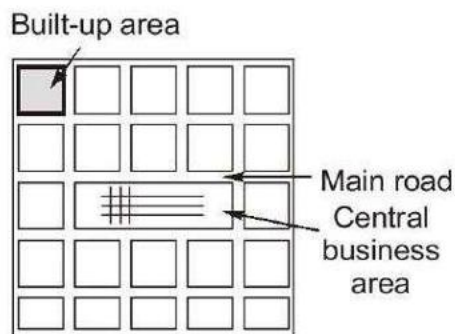
Advantages:

- ❖ The rectangular plots may be further divided into small rectangular blocks for construction of buildings placed back to back, having roads on their front.
- ❖ In this pattern has been adopted for the city roads.
- ❖ The construction and maintenance of roads of this pattern is comparatively easier.

Limitations:

This pattern is not very much convenient because at the intersections, the vehicles face each other.

Example: Chandigarh has rectangular pattern



(a) Rectangular or Block pattern

2. Radial or star and block pattern

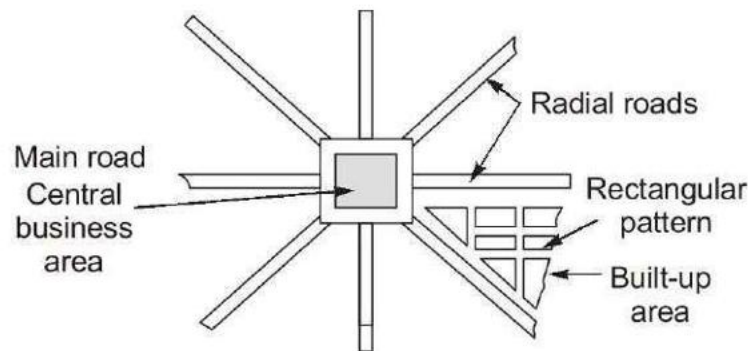
In this pattern, the entire area is divided into a network of roads radiating from the business outwardly. In between radiating main roads, the built-up area may be planned with rectangular block.

Advantage:

- ❖ Reduces level of congestion at the primary bottleneck location.
- ❖ Prevents traffic from accessing local flow routes in the direction of the event venue that operate in favor of degress traffic flow.
- ❖ If one is block then other side traffic can move.
- ❖ Vehicles face each other less than block pattern.

Limitations:

- ❖ Proves particularly effective if two-lane ramp traffic does not have to merge at downstream end of ramp.
- ❖ Safety appurtenances such as guide rail transitions, crash attenuators, and post support bases have not been designed to provide adequate protection at hazardous locations from the opposite direction of travel



(b) Radial or Star and Block pattern

3. Radial or star and circular pattern

In this system, the main radial roads radiating from central business area are connected together with concentric roads. In these areas, boundary by adjacent radial roads and corresponding circular roads, the built-up area is planned with a curved block system.

Advantages:

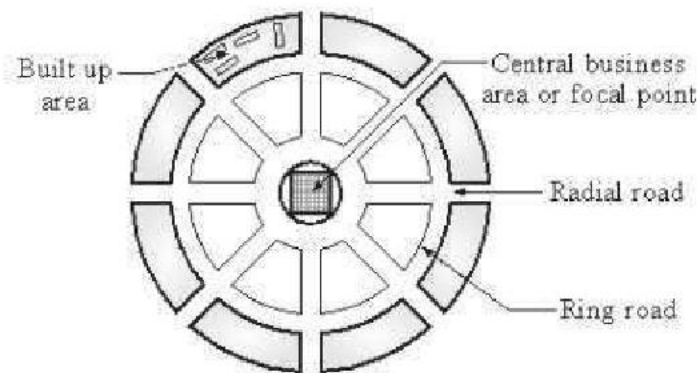
- ❖ At traditional intersections with stop signs or traffic signals, some of the most common types of crashes are right-angle, left-turn, and head-on collisions. These types of collisions can be severe because vehicles may be traveling through the intersection at high speeds. With circular pattern, these types of potentially serious crashes essentially are eliminated because vehicles travel in the same direction.

- ❖ Installing circular pattern in place of traffic signals can also reduce the likelihood of rear-end crashes.
- ❖ Removing the reason for drivers to speed up as they approach green lights and by reducing abrupt stops at red lights.
- ❖ Because roundabouts improve the efficiency of traffic flow, they also reduce vehicle emissions and fuel consumption.

Limitations:

- ❖ Center lines of roads leading to circular pattern should be properly aligned with the central island.
- ❖ Approach roads should be sufficiently curved, far enough in advance of circular pattern, to reduce vehicle speeds of entering drivers.
- ❖ Islands separating the approach and exit lanes, known as splitter islands, should extend far enough to provide pedestrian refuge and to delineate the roundabout.
- ❖ Traffic signs, pavement markings, and lighting should be adequate so that drivers are aware that they are approaching a roundabout and that they should reduce their travel speed.
- ❖ For older drivers declines in vision, hearing, and cognitive functions, as well as physical impairments, may affect some older adults' driving ability. Intersections can be especially challenging for older drivers.

Example: Intersection with traffic signals converted to a circular pattern in Asheville, North Carolina



Radial or star and circular pattern

4. Radial or star and grid pattern

Change in direction, and because street patterns are the most enduring physical element of any layout, it could potentially contribute to systematic site planning and, consequently, deserves a closer look. Though the network is entirely interconnected, north-south movement becomes

circuitous, indirect, and inconvenient, making driving an unlikely choice and vividly illustrating that interconnectedness by itself is insufficient to facilitate movement.

Advantages:

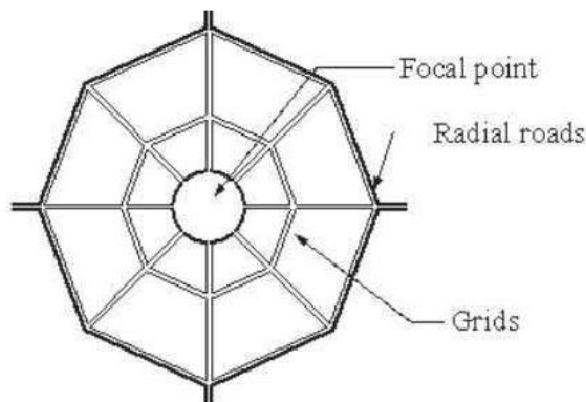
- ❖ Keep vehicular traffic safe with a high proportion of 3-way intersections.
- ❖ Reduce cut-through traffic by similar or other means.
- ❖ Improve traffic flow in both directions using Savannah's cellular structure.
- ❖ Improve land use efficiency and unit density.

Limitations:

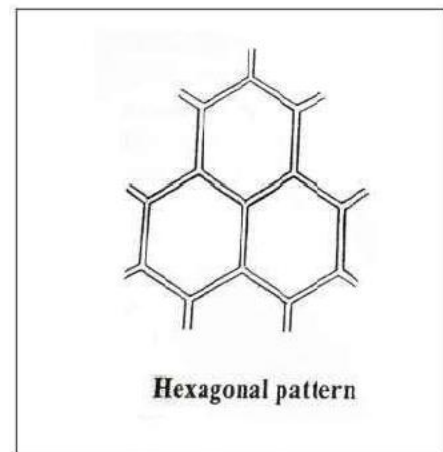
- ❖ Islands separating the approach and exit lanes, known as splitter islands, should extend far enough.
- ❖ Traffic signs, pavement markings, and lighting should be adequate so that drivers are aware that they should reduce their travel speed.

Examples:

The Nagpur road plan formulae were prepared on the assumption of Grid pattern.



Radial or star and grid pattern



4.Hexagonal pattern

In this pattern, the entire area is provided with a network of roads formatting hexagonal figures. At each corner of the hexagon, three roads meet the built-up area boundary by the sides of the hexagons is further divided in suitable sizes.

Advantages:

Three roads meet the built-up area boundary by the sides of the hexagons.

Limitations:

Traffic signs, pavement markings, and lighting should be adequate so that drivers are aware that they should reduce their travel speed

HIGHWAYS ALIGNMENT

Introduction

The position or the layout of the center line of the highway on the ground is called the alignment. It includes horizontal alignment i.e., straight path, horizontal deviation and curves and vertical alignment i.e., changes in gradient and vertical curves.

If any improper alignment present, leads to increase in

- Construction cost
- Maintenance cost
- Vehicle operation cost
- Accident rate

Requirements of an Ideal Alignment

An ideal alignment between two stations should offer maximum utility by serving maximum population and products and also should possess following requirements

- **Short:** it is desirable to have a short alignment between two stations. A straight path between the two terminals would provide this.
- **Easy:** it is should be easy to construct and maintain the road with minimum problems and also the alignment should be easy for vehicle to operate with easy gradient and curves.
- **Safe:** it should be safe enough for construction and maintenance from the view point of stability of natural hill slopes, embankment and cut slopes. It should be safe for the traffic operation with safe geometric features.
- **Economical:** The road alignment could be considered economical only if the total cost including initial cost, maintenance cost and vehicle operation cost is lowest.

Factors Controlling Alignment

The various factors which control the highway alignment are

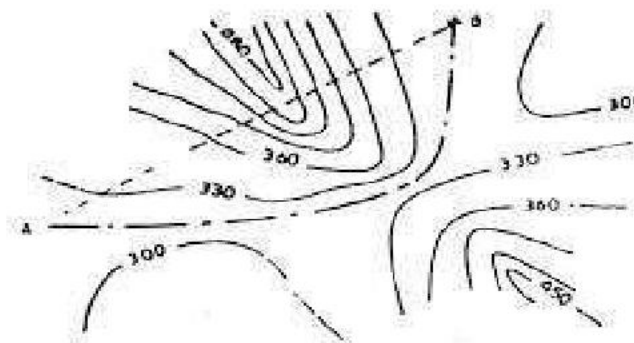
1. **Obligatory points**
2. **Traffic**
3. **Geometric design**
4. **Economics**
5. **Other considerations**

1. Obligatory Points: there are control points governing the alignment of the highways. These control points may be divided broadly into two categories.

- Points through which the alignment is to pass

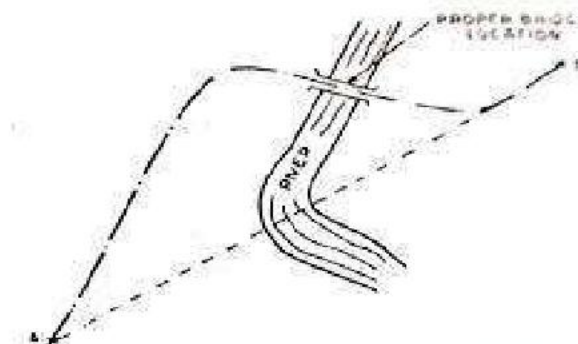
This may cause the alignment to often deviate from the shortest or easiest path. The various examples of this category may be bridge site, intermediate town, a mountain pass or quarry.

Case 1: When it is necessary to cross hill range, mountains the various alternatives are to cut a tunnel or to go around the hill. This suitability depends on many factors like site condition, topography, cost consideration etc. Figure (a) shows how the straight alignment AB is deviated along the hill side pass, thus avoiding a tunnel or heavy cutting.



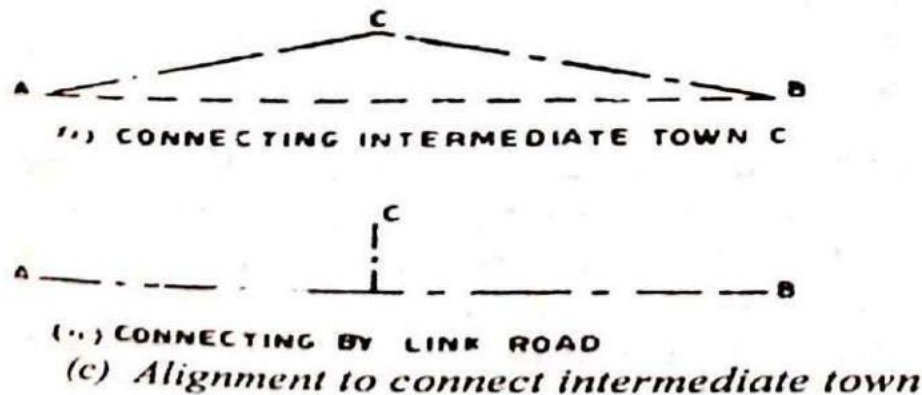
(a) Alignment along a hill side pass

Case 2: the road bridge across a river can be located at a place where the river has straight and permanent path and where there is a bend in the river, also the selected location of the bridge should be such that the abutment and pier can be properly constructed. Figure (b) shows that the straight alignment between stations A and B which crosses the river at the bend is not a suitable location and hence the alignment is to deviated along the path shown (by dash lines).



(b) Alignment to suit proper location of bridge

Case 3: when connecting a road network between two stations, it always beneficial to provide a link road or to connect the proposed alignment through nearby station. The straight alignment AB may be shifted along line ACB, as shown in figure (c) in order to connect the intermediate station C. It is also possible to connect the station C with a link road as shown in figure, thus avoiding the deviation of the straight alignment.



- Points through which the alignment should not pass

This may also make it necessary to deviate from the proposed shortest alignment due to the obligatory points through which alignment should not pass such as religious places (temples, church, mosque and grave), very costly structure, unsuitable land (lakes, ponds, marshy soil)

2. Traffic

The alignment should suit traffic requirements. Origin and destination study should be carried out in the area and the desire lines be drawn showing the traffic flow. The new alignment should keep in view of desired lines, flow patterns and future trends.

3. Geometric design

Geometric design factors such as gradient, radius of curve and sight distance, overtaking sight distance, ruling gradient on hilly region also would govern the final alignment of the highway. As far as possible while aligning a new road, the gradient should be flat and less than the ruling or design gradient.

4. Economics

The alignment finalized based on the above factors should be economical compared to other alignment. While working out the economics, the factors to be considered are initial construction cost, regular and periodic maintenance cost of the road and vehicle operation cost in future years. It is essential to work out overall economics based on 'life cycle cost' of the road project and not consider the initial cost of the road project only.

5. Other consideration

Factors like drainage consideration, hydrological factors, water table, seepage flow, high flood level, political considerations and monotony also affect in deciding the alignment. Straight road of very long stretch may be monotonous for driving. Hence, after a few kilometers of straight road, it may be desirable to have a slight bend or to provide some road side amenities, to break the monotony and keep the driver alert.

Special Considerations for Hilly Area Alignment

In hill roads additional care has to be given for

- **Stability**
- **Drainage**
- **Special geometric standards of hill roads**
- **Resisting length.**

Stability

While aligning hill roads, special care should be taken to align the road along the side of the hill which is stable. A common problem in hill roads is that of landslides. The cutting and filling of earth to construct roads on hill side causes steepening of existing slopes and this affects its stability of the hill slopes.

Drainage

Numerous hill side drains should be provided for adequate drainage facility across the road. But the cross drainage structure being costly, attempts should be made to align the road in such a way that the number of very expensive cross drainage structures is kept minimum.

Geometric standard of hill roads

Different sets of geometric design standards are followed on hill roads with reference to gradient, curves and speed and they consequently influence the sight distance, radius of curve and other related features. The route should enable the ruling gradient to be attained in most of the length, minimizing steep gradients, hair pin bends and needless rise and fall.

Resisting length

The resisting length of a road may be calculated from the total work to be done to move the loads taking the horizontal length, the actual difference in levels between the two stations and the sum of ineffective rise and fall in excess of floating gradient. In brief, the resisting length of the alignment should be kept as low as possible.

Engineering Surveys for Highway Alignment

The stages of engineering surveys for highway locations are

- 1. Map study**
- 2. Reconnaissance survey**
- 3. Preliminary surveys**
- 4. Final location and detailed surveys**

1. Map study

- ❖ By careful study of topographical map, it is possible to have an idea of several possible alternate routes so that further details of these may be studied later at the site.
- ❖ The features like river, hills valleys, and counter intervals can be observed.
- ❖ By knowing these feature it can fairly assign the alignment avoiding valleys, lakes and possible location of bridge (avoiding sharp turns etc.).
- ❖ It is also possible to suggest permissible gradient considering counter intervals.

2. Reconnaissance

It is to examine the general character of the area for deciding the most feasible routes for detailed studies. A field survey party may inspect a fairly broad stretch of land along the proposed alternative routes of the map in the field. Only few simple instruments like abney level tangent clinometers, barometer or GPS are used by the reconnaissance party to collect additional details rapidly. Some of the following details are collected

- ❖ Valleys, ponds, lakes, marshy land, hills, permanent structures and other obstruction along the route which are not available in the map.
- ❖ Approximate values of gradient, length of gradient and radius of curves of alternate alignments.
- ❖ Number and type of cross drainage structures, maximum flood level and natural ground water level along the probable routes.
- ❖ Sources of construction materials, water and location of stone quarries.
- ❖ When the alignment passes through hill, additional details like type of rocks, dip of strata, seepage flow.

3. Preliminary Survey

This survey can be done either by Conventional approach or aerial survey if the area is more.

The main objectives of preliminary survey are

- ❖ To survey the various alternate alignments proposed after the reconnaissance and to collect all the necessary physical information and details of topography, drainage and soil.
- ❖ To compare the different proposals in view of the requirements of a good alignment.
- ❖ To estimate quantity of earth work materials and other construction aspects and to work out the cost of alternate proposals.
- ❖ To finalize the best alignment from all consideration.

4. Final Location and Detailed survey

- ❖ The alignment finalized at the design office after the preliminary survey is to be located on the field by establishing the centre line.
- ❖ The centre line of the road finalized is to be translated on the ground during the location survey
- ❖ The centre line stakes are driven at suitable intervals say 50 m in plain and rolling terrains and at 20 m in hilly terrain.
- ❖ Temporary bench marks are fixed at intervals of about 250 m and at all drainage and under pass structures.
- ❖ The cross sectional levels are taken at very 50 – 100 m in plain terrain, 50 – 75m in rolling terrain, 50m in built-up area and 20m in hilly areas intervals.
- ❖ The cross section should be taken at curves and where there is a gradient change.
- ❖ The data collected during the detailed survey should be elaborate and complete for preparation of detailed plans, design and estimate of the project.

HIGHWAY GEOMETRIC DESIGN

Geometric design of highways deals with the following elements

- Cross section elements
- Sight distance considerations
- Horizontal alignment details
- Vertical alignment details
- Intersection elements

Design Control and Criteria

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

1. **Design speed**
2. **Topography or terrain**
3. **Traffic factors**
4. **Design hourly volume and capacity**
5. **Environmental and other factors**

1. Design speed

The design speed is the most important factor controlling the geometric design elements of highways. The design speed is decided taking into account the overall requirements of the highway. In India, different standards have been assigned depending upon the importance of the class of road. Further the design speed standards are modified depending upon the terrain or topography. Design of almost every geometric design elements of a road is dependent on the design speed.

The recommended design speed is given in Table .

Table : Design speed in as per IRC (ruling and minimum)				
Type	Plain	Rolling	Hilly	Steep
NS&SH	100-80	80-65	50-40	40-30
MDR	80-65	65-50	40-30	30-20
ODR	65-50	50-40	30-25	25-20
VR	50-40	40-35	25-20	25-20

2. Topography

The topography or the terrain conditions influence the geometric design of highways significantly. The terrains are classified based on the general slope of the country across the alignment as plain, rolling, mountainous and steep terrain. The design standards specified for different classes of roads are different depending on the terrain classification. For example the design speed of NH and SH on plain terrain with general cross slope of 10% is 100kmph whereas on rolling terrain with general slope of 10 to 25% is 80kmph and that on mountainous terrain with cross slope 25 to 60% is 50kmph.

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

3. Traffic factors

The factors associated with traffic that affects geometric design of roads are the vehicular characteristics and human characteristics of road users. It is difficult to decide the design vehicle or the standard traffic lane under the mixed traffic flow condition prevalent especially on urban roads of developing countries like India. This is a complex problem. However, it is often necessary to consider some standard vehicle as the design vehicle. The important human factors which affect traffic behavior include the physical, mental and psychological characteristics of drivers and pedestrians.

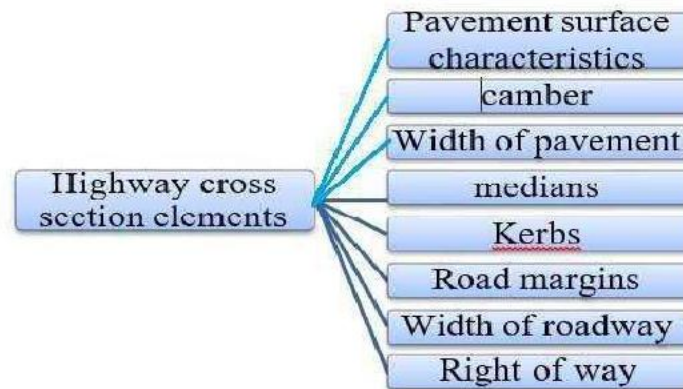
4. Design hourly volume and capacity

The traffic flow or volume keeps fluctuating with time, from a low value during certain off peak hours to the higher flow during the peak hours. It will be uneconomical to design the roadway facilities for the peak traffic flow or the highest hourly traffic volume. Therefore a reasonable value of traffic volume is decided for the design and this is called design hourly volume.

4.Environmental and other factors

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

Highway Cross Section Elements



1. Pavement Surface Characteristics

The pavement surface a) depends on the pavement type. The important characteristics of the pavement are

- (i) Friction
- (ii) Unevenness
- (iii) Light reflecting characteristics
- (iv) Drainage of surface water

(i) Friction

The friction or Skid resistance between vehicle tyre and pavement surface is one of the factors determining the operating speed and minimum distance required for stopping the vehicles. When vehicle negotiates a horizontal curve, the lateral friction developed counteracts the centrifugal forces and thus governs the safe operating speed. Frictional force is an important factor in the acceleration and retarding abilities of vehicles. 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 for calculating stopping sight distance and coefficient of lateral friction as 0.15 for horizontal curve design. Lack of adequate friction can cause skidding or slipping of vehicles.

- a. Skidding happens when the path traveled along the road surface is more than the circumferential movement of the wheels due to friction
- b. Slip occurs when the wheel revolves more than the corresponding longitudinal movement along the road.

Factors affecting friction

- ❖ 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)
- ❖ Speed of the vehicle
- ❖ Relative roughness
- ❖ Brake efficiency
- ❖ Load and tyre pressure
- ❖ Temperature of tyre and pavement
- ❖ Type of skid, if any

Below table shows the recommended value of 'f' by IRC

Speed, kmph	20 to 30	40	50	60	65	80	100 and above
Longitudinal friction coefficient value, f for SSD	0.40	0.38	0.37	0.36	0.36	0.35	0.35

(ii) Pavement Unevenness

Presence of undulations on the pavement surface is called pavement unevenness which results in

- (i) Increase in discomfort and fatigue to road users
- (ii) Increase in fuel consumption and tyre wear
- (iii) Increase in vehicle maintenance cost
- (iv) Reduction in vehicle operating speed increase in accident rate

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. It is measured by an instrument called Bump integrator (BI) in terms of unevenness index. 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.

The unevenness may be caused due to the following reasons

- (i) Inadequate or improper compaction of either the fill, subgrade or pavement layers or combination of these
- (ii) Use of improper construction machinery

- (iii) Use of inferior pavement materials
- (iv) Improper surface and subsurface drainage
- (v) Unscientific construction practices
- (vi) Poor maintenance practices
- (vi) Localized failures

(iii) Light Reflecting Characteristics

- (v) Night visibility depends upon the color and light reflecting characteristics of the pavementsurface
- (vi) The glare caused by the reflection of head light is high on wet pavement surface than dry
- (vii) White roads have good visibility at night, but caused glare during day time.
- (viii) Black roads has no glare during day, but has poor visibility at night

(iv) 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.

2. Camber

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

- a. to prevent the entry of surface water into the pavement layers and subgrade soil
- b. To prevent the entry of water into the bituminous pavement layers
- c. To remove the rain water from the pavement surface as quickly as possible

The required camber of a pavement depends on

- ❖ Type of pavement surface
- ❖ The amount of rainfall

Types of camber

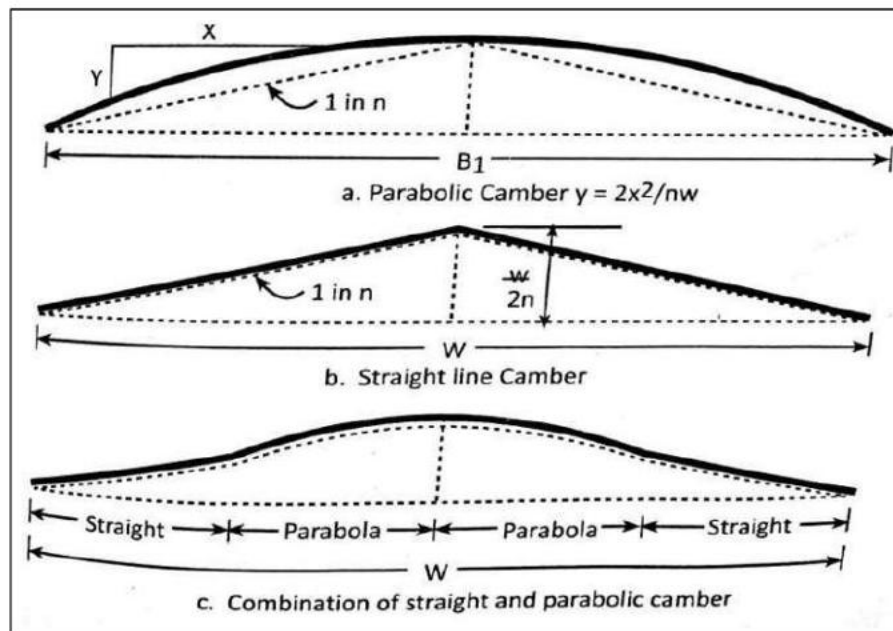
There are generally three types of the cambers

- Straight Camber
- Parabolic Camber
- Combination of straight and parabolic camber

Straight Camber: This type of camber is provided by meeting two straight surfaces at the crown. Crown is the central and top most point on the surface of the road. The edge shape produces inconvenience to the traffic so it is not used in general.

Parabolic Camber: Parabolic camber is provided by providing a parabolic shape to the surface of the road. It is also not used in general because it has steep slopes towards the edges, which can create the outward thrust to the vehicles.

Combination Camber: This camber is formed by use of the straight surfaces at the edges but parabolic surface at the centre. It is mostly used for the road construction because both the problem of the earlier two is solved if we use this camber.



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

S.No	Type of Road Surface	Range of camber in areas of	
		Heavy Rain fall	Low Rain fall
1	Cement concrete and high type bituminous surface	1 in 50 or 2.0%	1 in 60 or 1.7%
2	Thin bituminous surface	1 in 40 or 2.5%	1 in 50 or 2.0%
3	Water bound Macadam and gravel pavement	1 in 33 or 3.0%	1 in 40 or 2.5%
4	Earth road	1 in 25 or 4.0%	1 in 33 or 3.0%

Width of Pavement or 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 2.5m and the desirable side clearance for single lane traffic is 0.625 m. This requires minimum of lane width of 3.75 m for a single lane road (figure a) and 7m for two lane road (figure b). The desirable carriage way width recommended by IRC is given in Table

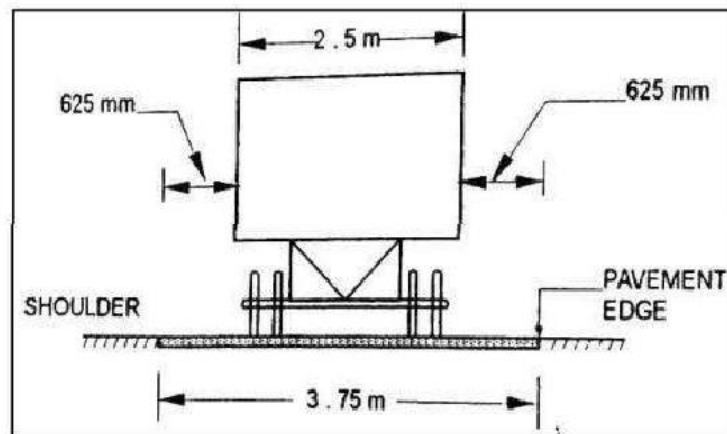


figure a-single lane road

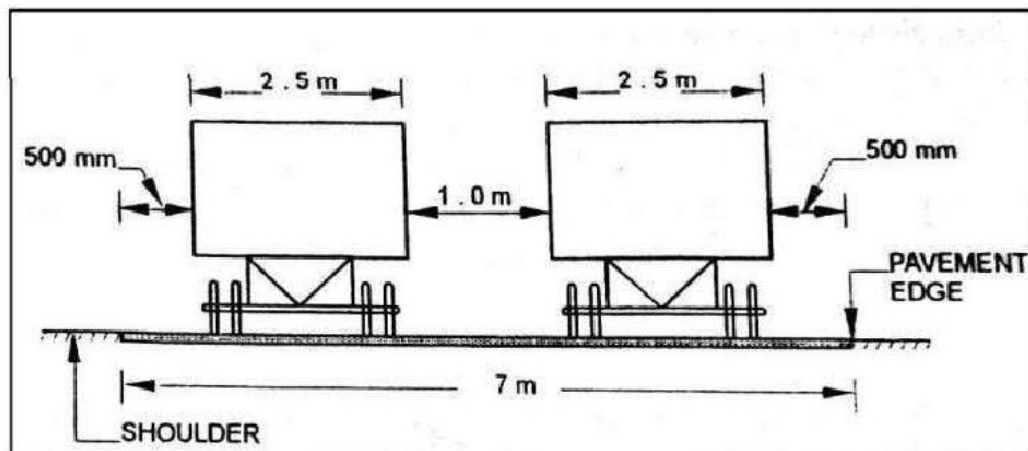
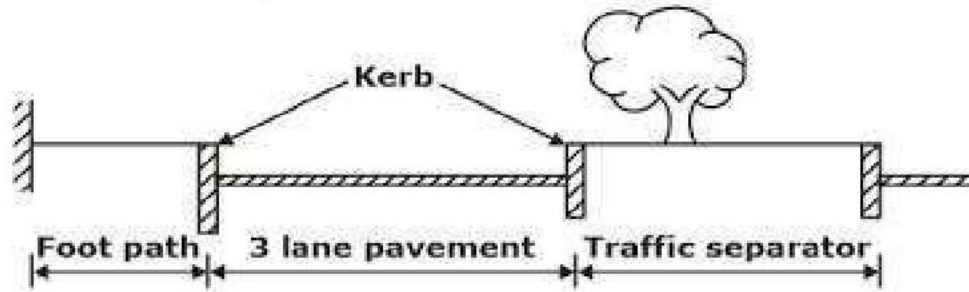


figure b - two lane road

Class Of Road	Width Of Carriageway (m)
Single lane road	3.75
Two lanes, without raised kerbs	7
Two lanes, with raised kerbs	7.5
Intermediate Carriageway	5.5
Multi-lane pavements	3.5 per lane

Medians or traffic separators



In highways with divided carriageway, a median is provided between two sets of traffic lanes intended to divide the traffic moving in opposite direction. The main function of the median is to prevent head on collision between vehicles moving in opposite directions on adjacent lanes. The traffic separators used may be in the form of pavement markings, physical dividers or area separators. The functions of medians are

- a) To channelize traffic into streams at intersections
- b) To shadow the crossing and turning traffic
- c) To segregate slow traffic
- d) To protect pedestrians

The IRC recommends a minimum desirable width of 5m for medians of rural highways, which may be restricted to 3m. The absolute minimum width of median in urban area is 1.2m and desirable minimum is 5m.

Kerbs

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

Different types of kerbs are

❖ **Low or mountable kerbs**

These types of kerbs are 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**

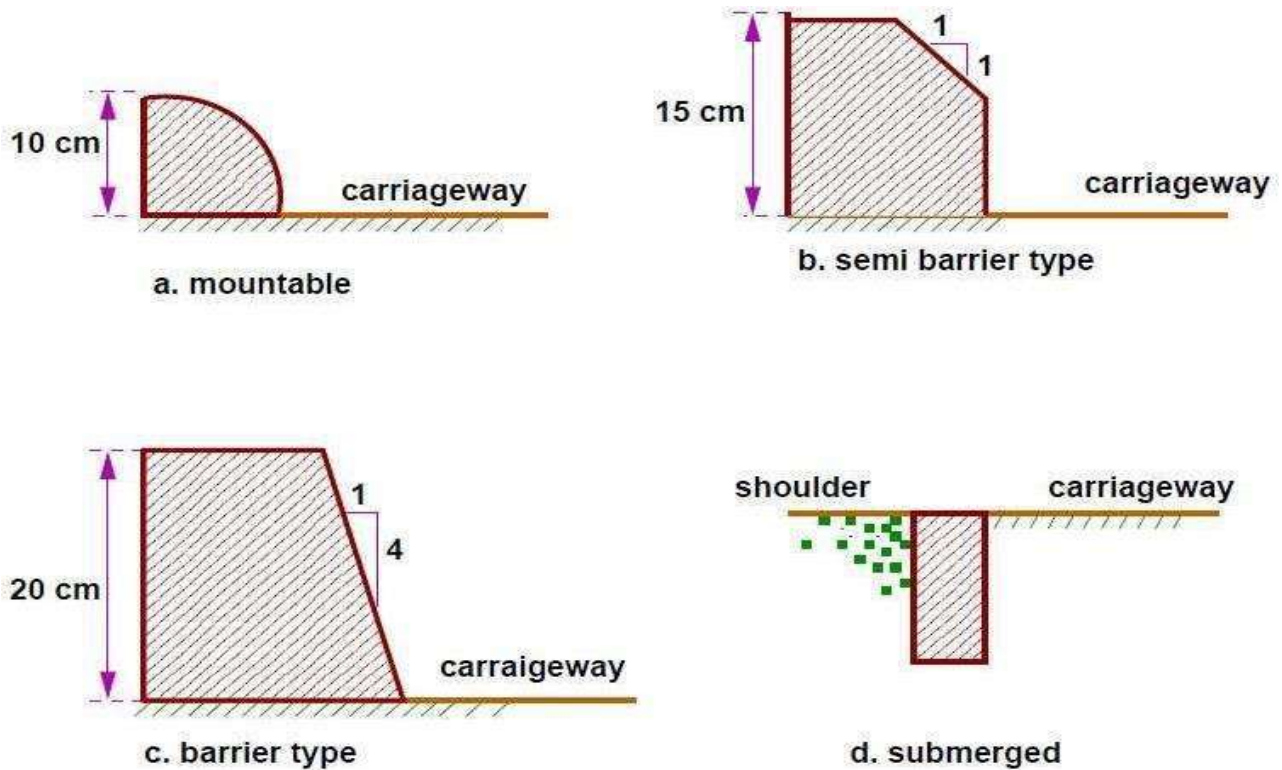
When the pedestrian traffic is high, these kerbs are provided. Their height is 15cm above the pavement edge. This type of kerb prevents encroachment of parking vehicles, but at acute emergency it is possible to drive over this kerb with some difficulty.

❖ Barrier type kerbs

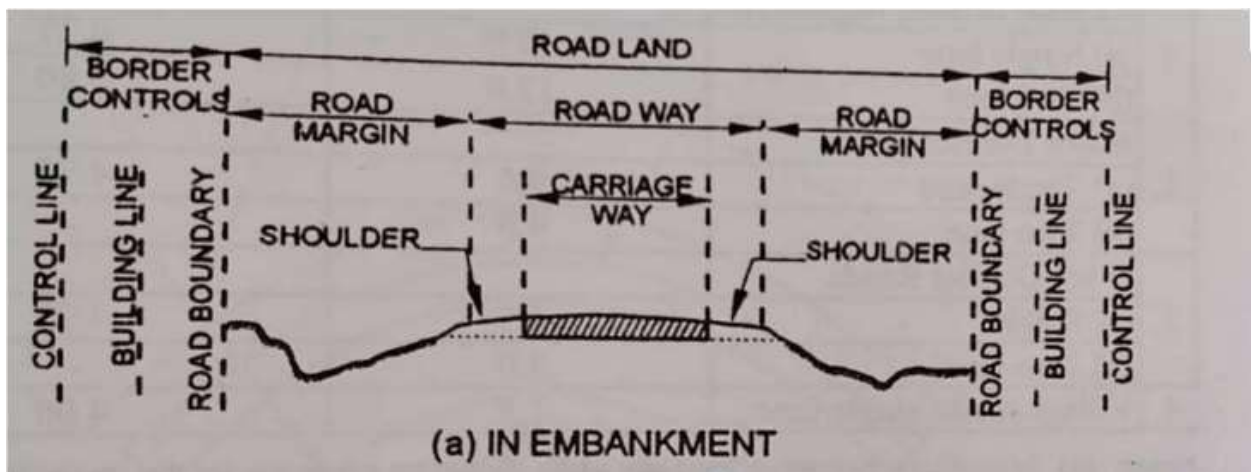
They are designed to discourage vehicles from leaving the pavement. They are provided when there is considerable amount of pedestrian traffic. They are placed at a height of 20 cm above the pavement edge with a steep batter.

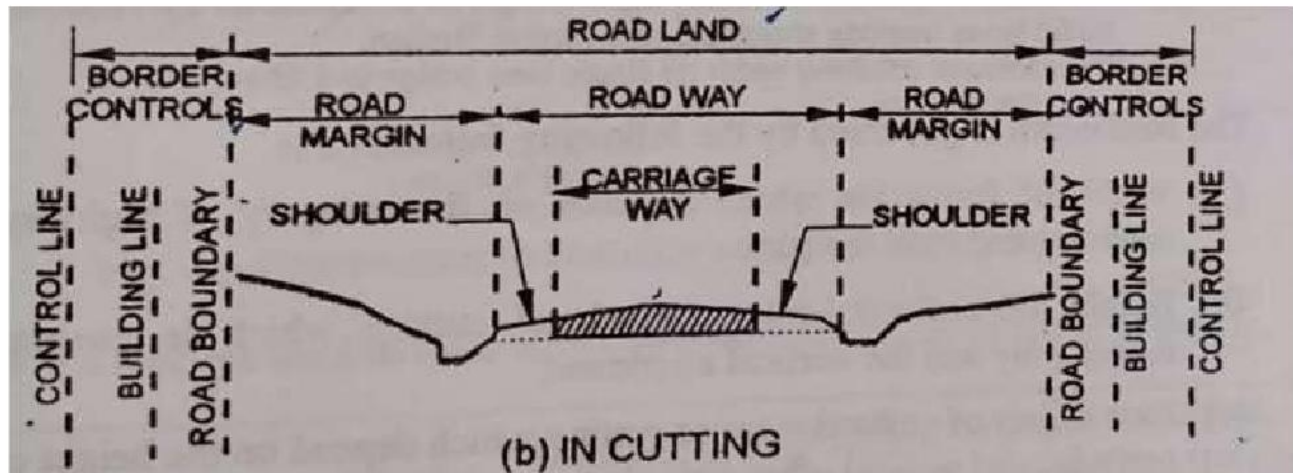
❖ Submerged kerbs

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





❖ Shoulder

Shoulders are provided along the road edge and are 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. A minimum width of 2.5 m is recommended by IRC for 2-lane rural highways in India.

The functions of shoulders are

- ❖ Structural ability and support to the edges of the flexible pavements
- ❖ Capacity of carriageway and the operating speeds of vehicles increases
- ❖ Serves as an emergency lane
- ❖ Acts as service lanes for vehicles that are disabled

❖ 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. As far as possible parallel parking should be allowed as it is safer for moving vehicles.

❖ 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 near the intersections is not affected by the bus-bay.

❖ 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.

❖ Guard rails

They are provided at the edge of the shoulder usually when the road is on an embankment. They serve to prevent the vehicles from running o the embankment, especially when the height of the fill exceeds 3m. Various designs of guard rails are in use. Guard stones painted in alternate black and white are usually used. They also give better visibility of curves at night under headlights of vehicles.

❖ Drive ways

Drive ways connect the highway with commercial establishment like fuel stations, servicestations etc. Drive ways should be properly designed and located, fairly away from an intersection. The radius of the drive way curve should be kept as large as possible, but the width of the drive way should be minimized to reduce the crossing distance of pedestrians.

❖ Lay byes

They are provided near public conveniences with guide maps to enable drivers to stop clear off the carriageway. It should normally be 3m width and at least 30m length with 15m end tapers on both sides.

❖ Frontage road

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

❖ Embankment slope

Embankment slope should be as flat as possible for the purpose of safe traffic movement and also for aesthetic reasons. For safety considerations, the desirable slope for the embankment is 1 in 3.

Width of Formation or Roadway

Width of formation or roadway is the sum of widths of pavement or carriageway including separators, if any and the shoulders. The width of roadway standardized by the IRC is given in table below



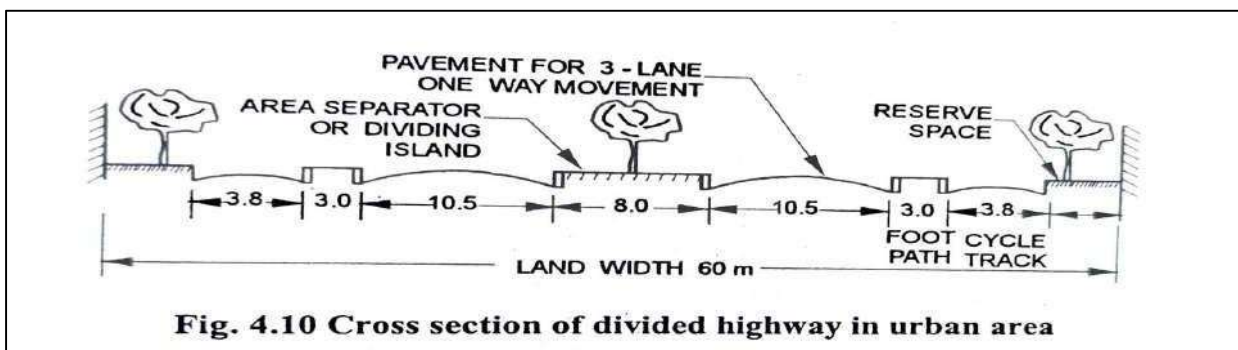
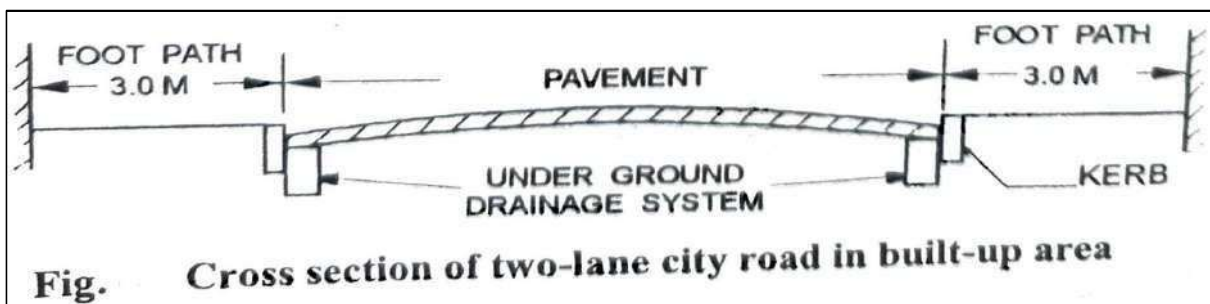
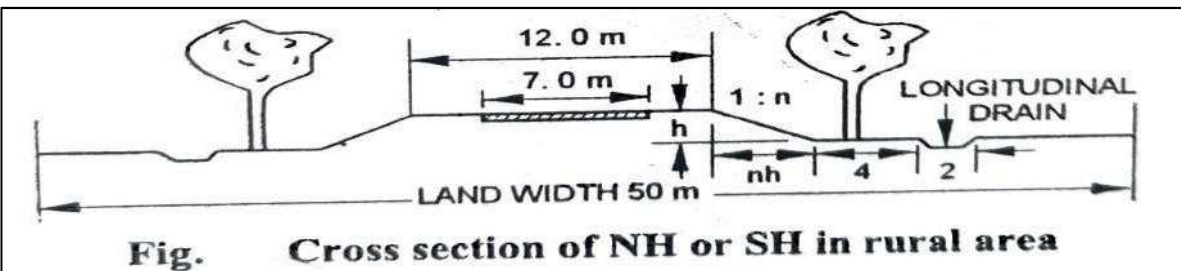
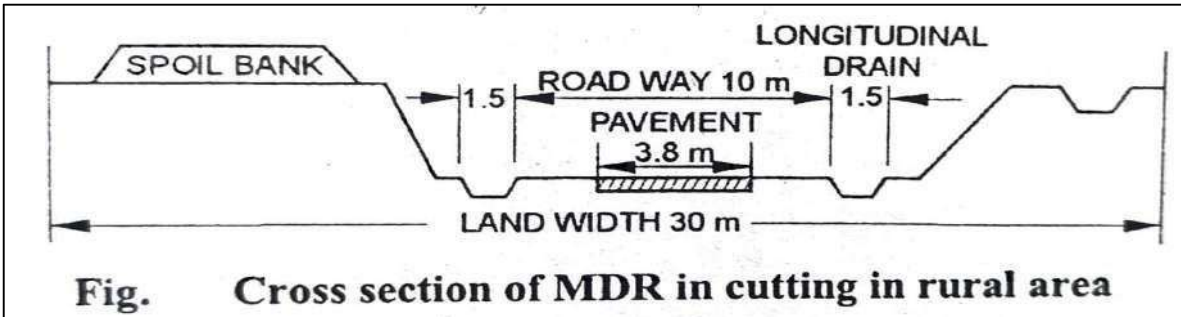
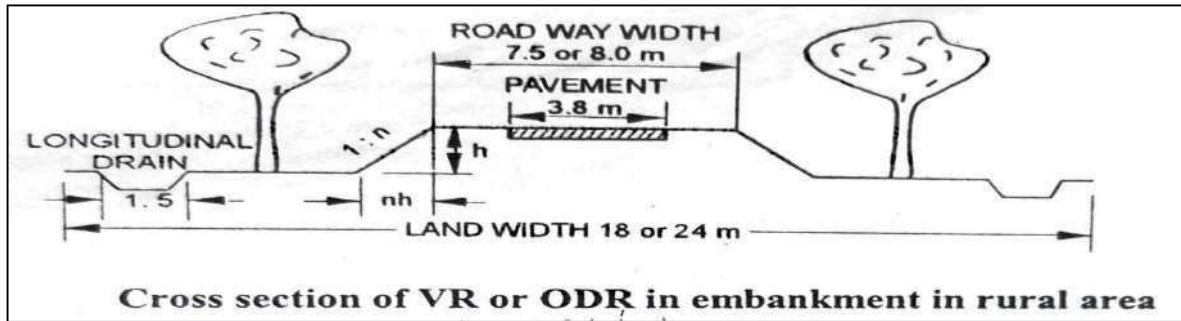
Sr. No	Road Classification	Roadway Width (m)	
		Plain and rolling terrain	Mountainous and steep terrain
1	National & State Highways		
	a) Single Lane	12	6.25
	b) Two lane	12	8.80
2	Major District Roads		
	a) Single Lane	9	4.75
	b) Two lane	9	-
3	Other District Roads		
	a) Single Lane	7.5	4.75
	b) Two lane	9	-
4	Village Roads ,Single Lane	7.5	4

Right of Way and Land Width

Right of way is the area of land acquired for the road, along its alignment. The width of the acquired land for right of way is known as land width and depends on the importance of the road and possible future development. To prevent 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.

Typical Cross Section of Roads

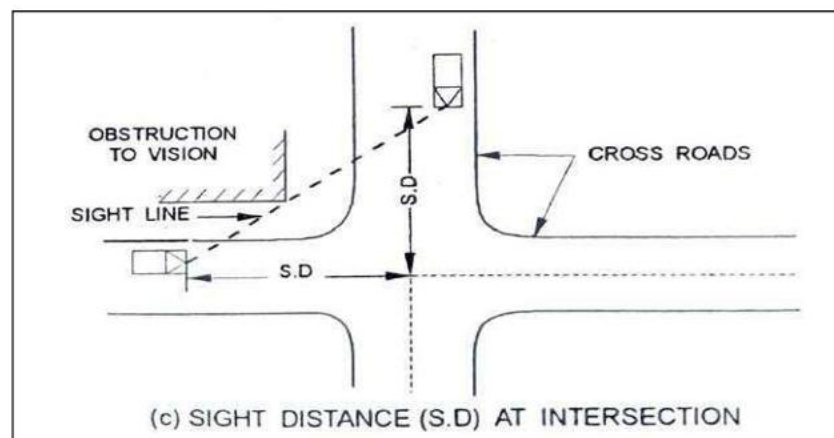
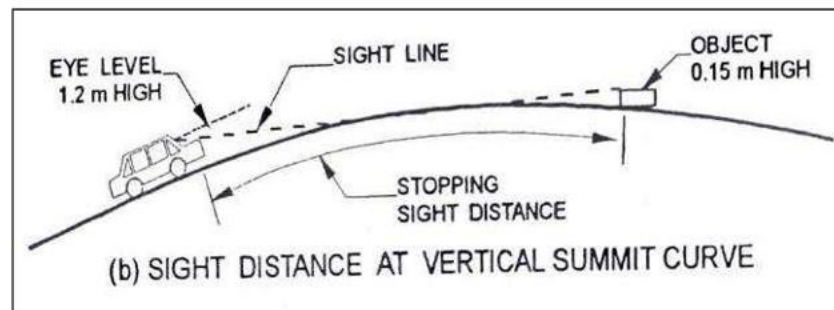
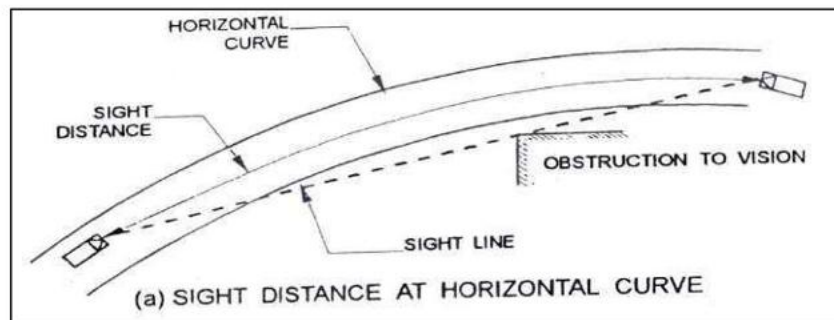


Sight Distance

It is the length of road visible ahead to the driver at any instance. Sight distance available at any location of the carriageway is the actual distance a driver with his eye level at a specified height above the pavement surface has visibility of any stationary or moving object of specified height which is on the carriageway ahead.

Restrictions to Sight Distance

- At horizontal curves, when the line of sight is obstructed by objects at the inner side of the curve (fig a)
- At a vertical curve the line of sight is obstructed by the road surface of the summit curve (figb)
- At an uncontrolled intersection when a driver from one of the approach roads is able to sight a vehicle from another approach road proceeding towards the intersection (fig c)



Types of Sight Distance

Three types of sight distances situations are considered in the design

1. Stopping sight distance (SSD) or Absolute minimum sight distance

2. Safe overtaking sight distance (OSD) or Passing sight distance

3. Safe sight distance for entering into uncontrolled intersections

1. Stopping sight distance (SSD)

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.

2. Safe overtaking sight distance

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

3. Safe sight distance for entering into uncontrolled intersections

Driver entering an uncontrolled intersection has sufficient visibility to enable him to take control of his vehicle and to avoid collision with another vehicle

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

❖ Intermediate sight distance

This is defined as twice the SSD. When OSD cannot be provided, intermediate sight distance (ISD) is provided to give limited overtaking opportunities to fast vehicles.

❖ Head – light sight distance

This is the distance visible to a driver during night driving under the illumination of the vehicle head lights. This sight distance is critical at up gradient and at ascending stretch of the valley curves

Stopping Sight Distance (SSD)

The minimum distance visible to a driver ahead on a highway at any spot should be of sufficient length to safely stop a vehicle travelling at design speed, without collision with any other obstruction. This is also called as non passing sight distance or absolute minimum sight distance.

The sight distance available to a driver traveling on a road at any instant depends on the following factors

- (i) Features of the road ahead
- (ii) Height of the driver's eye above road surface
- (iii) Height of the object above the road surface

The features of the road ahead which affects the sight distance are the horizontal alignment and vertical profile of the road, the traffic condition and the position of obstructions. For the purpose of measuring the stopping sight distance or visibility ahead, IRC has suggested the height of eye level of driver as 1.2m and the height of the object as 0.15m above the road surface.

Factors on Which Stopping Sight Distance Depends

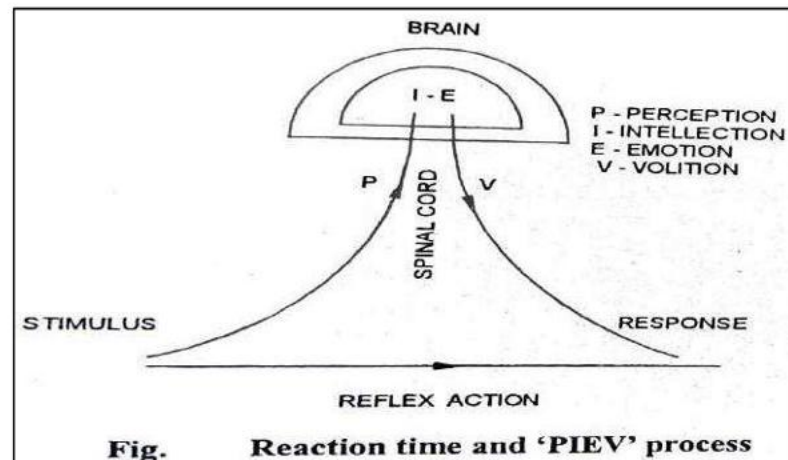
1. Total reaction time of the driver
2. Speed of vehicle
3. Efficiency of brakes
4. Frictional resistance between the road and the tyre
5. Gradient of the road, if any

1. Total reaction time

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 two parts: perception and brake reaction time. The perception time is the time required for a driver to realize that brakes must be applied. It is the time from the instant the object comes on the line of sight of the driver to the instant he realizes that the vehicle needs to be stopped. The brake reaction time depends on several factors including the skill of the driver, the type of the problems and various other environmental factors. The total reaction time may be explained with the help of PIEV theory.

PIEV theory

According to PIEV theory, the total reaction time of the driver is split into four parts



1. Perception time

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

2. Intellection time

It is the time required for the driver to understand the situation. It is also the time required for comparing the different thoughts, regrouping and registering new sensations.

3. Emotion time

It is the time elapsed during emotional sensations and other mental disturbance such as fear, angry or any other emotional feelings like superstition etc with reference to situation. It varies for different drivers.

4. Volition time

It is the time taken by the driver for the final action, such as brake application.

2. Speed of vehicle

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

3. 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 is less.

4. Frictional resistance between road and tyres

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.

5. 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.

Analysis of Stopping Sight Distance

The stopping sight distance of a vehicle is the sum of

1. The distance travelled by the vehicle at uniform speed during the total reaction time, t which is known as lag distance
2. The distance travelled by the vehicle after the application of the brakes, until the vehicle comes to a dead stop which is known as braking distance

Lag distance

If 'v' is the design speed in m/sec and 't' is the total reaction time of the driver in seconds, then

$$\text{Lag distance} = vt \text{ (m)}$$

If the design speed is V kmph, then the lag distance = $0.278Vt \approx 0.28Vt \text{ (m)}$

Braking distance

Assuming a level road, the braking distance may be obtained by equating the work done in stopping the vehicle and the kinetic energy of the vehicle moving at design speed. If the maximum frictional force developed is F (kg) and the braking distance is I (m), then work done

$$F * I = W * f * I$$

Kinetic energy of the vehicle of weight W moving at the design speed of v m/sec = $Wv^2/2g$

Hence

$$\frac{Wv^2}{2g} = \frac{Wl}{f + g}$$

Therefore **braking distance**,

$$l = \frac{v^2}{2g(f + g)}$$

Where l = braking distance, m

V = speed of vehicle, m/s

f = frictional coefficient

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

Stopping distance on level road

Stopping distance, SD = Lag distance + breaking distance

$$SD = \frac{V}{g} + \frac{v^2}{2g(f + g)}$$

If speed is V kmph, the stopping distance

$$SD = 0.278 \frac{V}{g} + \frac{V^2}{254(f + g)}$$

Stopping distance at slopes

$$SD = 0.278 \frac{V}{g} + \frac{V^2}{254(f \pm g)}$$

OVERTAKING SIGHT DISTANCE (OSD)

Over Taking Requirement

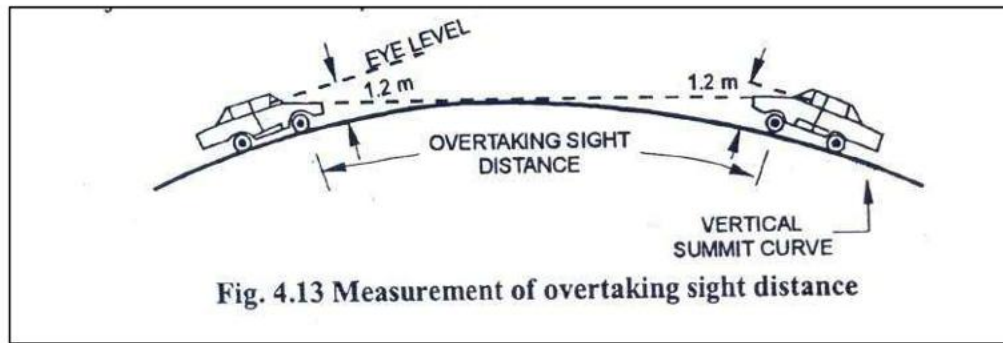
If all the vehicles travel along a road at the design speed, then theoretically there should be no need for any overtaking. In fact, all vehicles do not move at the design speed as each driver is free to travel at lower speeds and this is particularly true under Mixed Traffic conditions. It is necessary for fast moving vehicle to overtake or pass the slow-moving traffic.

The minimum distance open to the vision of the driver of a vehicle intending to overtake slow vehicle ahead with safety against the traffic of opposite direction is known as Minimum Overtaking Sight Distance (OSD) or Safe Passing Sight Distance

Overtaking Sight Distance (OSD) or Safe Passing Sight Distance

The OSD is the distance measured along the centre line of the road which a driver with his eye level at

1.2m above the road surface can see the top of an object 1.2m above the road surface



Factor Affecting OSD

> Speeds of

1) Overtaking Vehicle

2) Overtaken Vehicle

3) Vehicle coming from opposite direction

>>Distance between the overtaking and overtaken vehicles, the minimum spacing between vehicles depends on the speeds

>Skill and reaction time of the driver

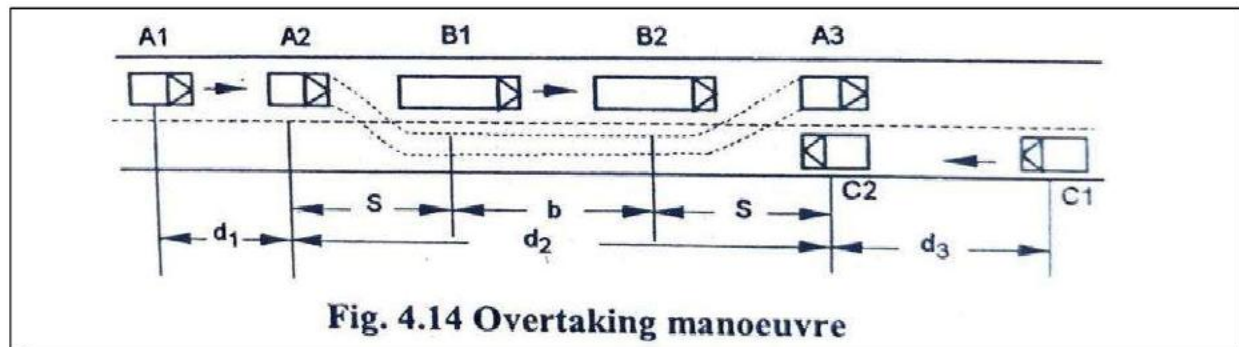
>Rate of acceleration of overtaking vehicle

> Gradient of the road

ANALYSIS OF OSD ON A 2 – WAY ROAD

Simple overtaking process on a 2 – lane highway with 2 – way traffic movement

Vehicle A travelling at the design speed v m/sec or V kmph desires to overtake another slow-moving vehicle B moving at a speed of v_b m/sec or V_b kmph. The vehicle A has to accelerate, shift to the adjacent right-side lane, complete the overtaking manoeuvre and return to the left lane, before oncoming vehicle C approaches the overtaking stretch.



The overtaking manoeuvre may be split up into 3 operations, thus dividing OSD into 3 parts d_1 , d_2 and d_3 .

- d_1 is the distance (m) travelled by the overtaking vehicle A during the reaction time, t (secs) of the driver from position A1 to A2 before starting to overtake the slow vehicle B
- d_2 is the distance travelled (m) travelled by the vehicle A during the actual overtaking operation during T (secs) from position A2 to A3
- d_3 is the distance (m) travelled by oncoming vehicle C during the actual overtaking operation of A during T (secs) from position C1 to C2.

Thus, on a 2-lane road with 2-way traffic the **OSD = $d_1 + d_2 + d_3$ in meters**

Assumptions made in the analysis

Assumptions made to calculate the values of d_1 , d_2 and d_3 (m) are given below:

- The overtaking vehicle A is forced to reduce its speed from the design speed v (m/sec) to V_b (m/sec) of the slow vehicle B and move behind it, allowing a space s (m), till there is an opportunity for safe overtaking operation
- When the driver of vehicle A finds sufficient clear gap ahead, decides within a reaction time t (sec) to accelerate and overtake the vehicle B, during which the vehicle A moves at speed v_b (m/sec) through a distance d_1 from position A1 to A2.
- The vehicle A accelerates and overtakes the slow vehicle B within a distance d_2 during the overtaking time, T (sec) between the position A2 to A3
- The distance d_2 is split up into three parts

a) Spacing, s (m) between A2 and B

b) Distance b (m) travelled by the slow vehicle B between B1 and B2 during the overtaking manoeuvre of A

c) Spacing (m) between B2 and A3

- During this overtaking time T (sec), the vehicle C coming from opposite direction travels through a distance d_3 from position C1 to C2

Determination of the components of OSD

a) From position A1 to A2, the distance, d_1 (m) travelled by overtaking vehicle A, at the reduced speed v_b (m/sec) during the reaction time, t (sec) = $v_b t$ (m). The IRC suggests that this reaction time Y of the driver

may be taken as 2.0 sec as an average value, as the aim of the driver is only to find an opportunity to overtake. Therefore

$$d_1 = 2v_b$$

b) From position A2, the vehicle A starts accelerating, shifts to the adjoining lane, overtakes the vehicle B, and shifts back to its original lane ahead of B in position A3 during the overtaking time, T (sec). The straight distance between position A2 and A3 is taken as d_2 (m), which is further split into three parts, viz.,

$$d_2 = (s + b + s)$$

c) The minimum distance between position A2 and B1 may be taken as the minimum spacing s (m) between the two vehicles while moving with the speed v_b (m/sec). The minimum spacing between vehicles depends on their speed and is given by empirical formula

$$s = (0.7 v_b + 6)$$

d) Now the time T depends on speed of overtaken vehicle B and the average acceleration a (m/sec²) of overtaking vehicle A. The overtaking time T (sec) may be calculated by equating the distance d_2 to $(v_b T + \frac{1}{2} a T^2)$ using the general formula for the distance travelled by a uniformly accelerating body with initial speed v_b m/sec and a is the average acceleration during overtaking in m/sec²

$$d_2 = (v_b T + 2s)$$

e) The distance travelled by vehicle C moving at design speed v (m/sec) during the overtaking operation of vehicle A i.e. during time T (sec) is the distance d_3 (m) between positions C1 to C2. Hence,

$$d_3 = v T \text{ (m)}$$

In m/sec units

$$OSD = (d_1 + d_2 + d_3) = (v_b t + v_b T + 2s + vT)$$

Here

v_b = initial speed of overtaking vehicle, m/s

t = reaction time of driver = 2 sec

V = speed of overtaking vehicle or design speed, kmph

$$\square = \sqrt{\frac{4s}{a}}$$

s = spacing of vehicles = $(0.7 v_b + 6)$

a = average acceleration during overtaking, m/sec.

In kmph units

$$OSD = 0.28 V_b t + 0.28 V_b T + 2s + 0.28 V T$$

Here V_b = initial speed of overtaking vehicle, kmph

t = reaction time of driver = 2 sec

V = speed of overtaking vehicle or design speed, kmph

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

A = average acceleration during overtaking, kmph

In case the speed of overtaken vehicle (v_b or V_b) is not given, the same may be assumed as 4.5 m/sec or 16 kmph less than the design speed of the highway.

Therefore, $v_b = (v - 4.5)$ m/sec

$V_b = (V - 16)$ kmph

where v is the design speed in m/sec

V is the design speed in kmph.

The acceleration of the overtaking vehicle varies depending on several factors such as the make and model of the vehicle, its condition, load and the speed; actual acceleration also depends on the characteristics of the driver. The average rate of acceleration during overtaking manoeuvre may be taken corresponding to the design speed.

Maximum overtaking acceleration at different speeds

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

CRITERIA FOR SIGHT DISTANCE REQUIREMENT ON HIGHWAY

Absolute Minimum Sight Distance

Safe stopping distance, SSD for the design speed is the absolute minimum sight distance and this should be made available all along the road stretches irrespective of the category of road. If on any road stretch SSD is not available due to any reason such as obstruction to vision, immediate steps should be taken to either remove the obstruction to the sight line or install suitable regulatory signs to specifying the speed limit along the appropriate warning signs. On horizontal curves the obstruction on the inner side of the curve should be cleared to provide the required set back distance such that the absolute minimum sight distance is invariably available.

Overtaking Zones

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

The width of carriageway and the length of overtaking zone should be sufficient for safe overtaking. Sign posts should be installed at sufficient distance in advance to indicate the start of the overtaking zones, this distance may be equal to

- $(d_1 + d_2)$ for one-way roads
- $(d_1 + d_2 + d_3)$ for two-way roads

The minimum length of overtaking zone = 3 (OSD)

The desirable length of overtaking zones = 5 (OSD)

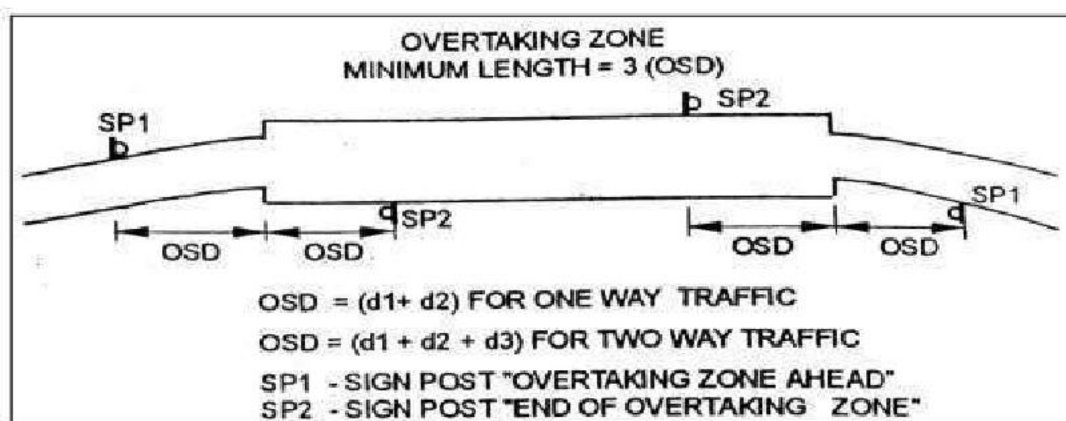
Sight Distance at Uncontrolled Intersections

It is important that on all approaches of intersecting roads, there is a clear view across the corners from a sufficient distance so as to avoid collision of vehicles. The design of sight distance at intersection may be based on three conditions Enabling the approaching vehicle to change speed

Intermediate Sight Distance

At stretches of the road where requires OSD cannot be provided, as far as possible intermediate Sight Distance ISD equal to twice SSD may be provided. The measurement of the ISD may be made assuming both the height of the eye level of the driver and the object to be 1.2 metres above the road surface.

Therefore **ISD = 2 SSD**



DESIGN OF HORIZONTAL ALIGNMENT

General

Often changes in the direction are necessitated in highway alignment due to various reasons such as topographic considerations, obligatory points. The geometric design elements pertaining to horizontal alignment of highway should consider safe and comfortable movement of vehicles at the designated design speed of the highway. It is therefore necessary to avoid sudden changes in direction with sharp curves or reverse curves which could not be safely and conveniently negotiated by the vehicles at design speed. Improper design of horizontal alignment of roads would necessitate speed changes resulting in higher accident rate and increase in vehicle operation cost.

Various design elements to be considered in the horizontal alignment are design speed radius of circular curve, type and length of transition curves, super elevation, widening of pavement on curves and required set-back distance for fulfilling sight distance requirements.

Design Speed

The design speed is the main factor on which geometric design elements depends. In other words, the geometric details of a highway mainly depend on the design speed. All the important geometric elements such as sight distances, radius of horizontal curve, length of horizontal transition curve, rate of super elevation, extra widening of pavement at horizontal curve, length of summit and valley curves are dependent on the design speed.

The design speed of roads depends upon

- 1) Class of the Road
- 2) Terrain

The speed standards of a particular class of road thus depends on the classification of terrain through which it passes. The terrains have been classified as plain, mountainous and steep, depending on the cross slope of the country as given in table below

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

Two values of design speeds are considered at the design stage of highway geometries namely,

- 1) Ruling design speed
- 2) Minimum design speed

As a general rule, attempt should be made to design all the geometric element of the highway for the 'ruling design speed'. This is because ruling design speeds are guiding criteria for the geometric design. However, 'minimum design speed' may be accepted where site conditions or economic considerations warrant.

The ruling design speeds suggested for the National and State Highways in India passing through plain terrain is 100 kmph and through rolling terrain is 80kmph and minimum design speed values standardized by the IRC for of roads on different terrains in rural (non-urban) areas are given in Table below

Road Classification	Design Speed in kmph							
	Plain		Rolling		Mountainous		Steep	
	Ruling	Min.	Ruling	Min.	Ruling	Min.	Ruling	Min.
Expressway	120	100	100	80	80	60	80	60
NH and SH	100	80	80	65	50	40	40	30
MDR	80	65	65	50	40	30	30	20
ODR	65	50	50	40	30	25	25	20
VR	50	40	40	35	25	20	25	20

The recommended design speeds for different classes of urban roads

- 1) Arterial Roads: 80 Kmph
- 2) Sub-Arterial Roads: 60 Kmph
- 3) Collector Streets: 50 Kmph
- 4) Local Streets: 30 Kmp

SUPERELEVATION

In order to counteract the effect of centrifugal force and to reduce the tendency of the vehicle to overturn or skid, the outer edge of the pavement is raised with respect to the inner edge, thus providing a transverse slope throughout the length of the horizontal curve. This transverse inclination to the pavement surface is known as **SUPER ELEVATION or CANT or BANKING**.

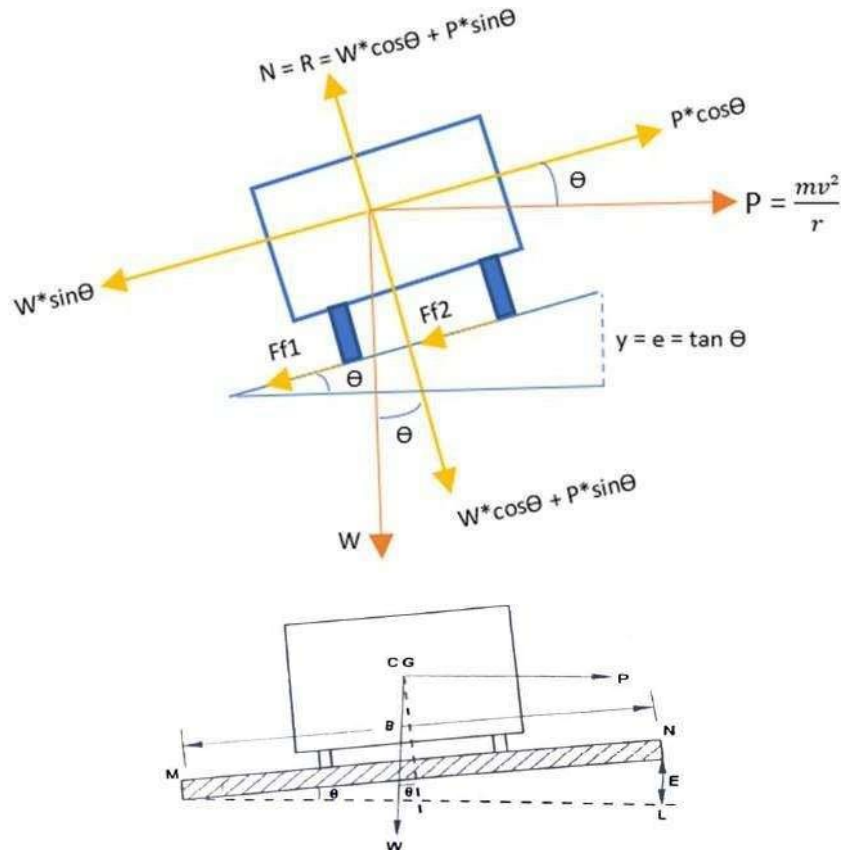


Fig. Superelevated pavement section

The rate of super elevation, 'e' is expressed as the ratio of the height of outer edge with respect to the horizontal width.

$$e = \frac{E}{B} = \tan \theta = \frac{h \tan \theta_1}{B} = \frac{h}{B} \tan \theta_1$$

E – Relative elevation of the outer edge

B – Width of the pavement

The general equation for design of super elevation is given by

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

e = rate of super elevation = $\tan \theta$

f = design value of lateral friction coefficient = 0.15

v = speed of the vehicle, m/sec

R = radius of the horizontal curve, m

g = acceleration due to gravity = 9.8 m/sec²

If the speed of the vehicle is given in kmph then the equation is

$$e + f = \frac{(0.0018 V)^2}{0.0001 R} = \frac{V^2}{R}$$

The maximum value of super elevation is limited to 7% or 0.07 and the minimum value of lateral friction of coefficient f for highway is 0.15

In some situations, particularly at, some intersections it is not possible to provide super elevation and in such cases the friction counteracts the centrifugal force fully. The allowable speed of vehicle negotiating a turn should be restricted to the condition

$$V = \sqrt{R}$$

The super elevation depends upon

- 1) Radius of the curve R,
- 2) Speed of the vehicle V
- 3) The coefficient of lateral friction f

Steps for Super Elevation Design

The steps for the design of super elevation in India from practical considerations (as per the IRC Guidelines) are given below:

- 1) The super elevation is calculated for 75% of design speed (0.75 v m/sec or 0.75 V kmph), neglecting the friction

$$e = \frac{(0.0018 V)^2}{R} = \frac{V^2}{R}$$

$$e \approx \frac{(0.0018 V)^2}{R}$$

- 2) If the calculated value of 'e' is less than 7% or 0.07 the value so obtained provided. If the value of 'e' as per the above equation exceeds 0.07 then provide the maximum super elevation equal to 0.07 and proceed with steps 3 or 4
- 3) Check the coefficient of friction developed for the maximum value e = 0.07 at the full value of design speed, v m/sec or V kmph

$$f = \left(\frac{V^2}{Rg} - e \right) \text{ or } \left(\frac{V^2}{Rg} - e \right)$$

If the value of f thus calculated is less than 0.15, the super elevation of 0.07 is safe for the design speed and this is accepted as the design super elevation. If not, either the radius of the horizontal curve has to be increased or the speed has to be restricted to the safe value which will be less than the design speed. The restricted speed or the allowable speed is calculated as given in step 4

- 4) The allowable speed or restricted speed (V_a m/sec or V_a kmph) at the cml ' is calculated by considering the design coefficient of lateral friction and the maximum super elevation. The safe allowable speed

$$V_a = \sqrt{g \cdot R \cdot (f + e)} = \sqrt{g \cdot R \cdot (f + e)} \text{ m/s or } \sqrt{g \cdot R \cdot (f + e)} \text{ kmph}$$

If the allowable speed, as calculated above is higher than the design speed design, then the design is adequate and provide a super elevation of 'e' equal to 0.07. If the allowable speed is less than the design speed, the speed is limited to the allowable V_a kmph calculated above. If the allowable speed V_a is less than the design speed V , appropriate warning signs and speed limit signs are to be installed.

Attainment of Super elevation in the field

Introduction of super elevation on a horizontal curve in the field is an important feature in construction. The road cross section at the straight portion is cambered with the crown at the center of the pavement and sloping down towards both the edges. But the cross section on the portion of circular curve of the road is super elevated with a uniform tilt sloping down from the outer edge of the pavement up to inner edge.

Thus, the crowned camber sections at the straight before the start of the transition curve should be changed to a single cross slope equal to the desired superelevation at the beginning of the circular curve. This change may be conveniently attained at a gradual and uniform rate through the length of horizontal transition curve. The full superelevation is attained by the end of transition curve or at the beginning of the circular curve.

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

- Elimination of crown of the cambered section
- Rotation of pavement to attain full superelevation

EXTRA WIDENING OF PAVEMENT ON HORIZONTAL CURVES

Objectives

- 1) An automobile such as car, bus or truck has a rigid wheel base and only the front wheels can be turned.
When the vehicle takes a turn to negotiate a horizontal curve, the rear wheels do not follow the same path as that of the front wheels. This phenomenon is called 'off tracking'. Normally at low speeds and up to the design speed when no lateral slipping of rear wheels take place, the rear wheels follow the inner path on the curve as compared with those of the corresponding front wheels. This means that if inner front wheel takes a path on the inner edge of a pavement at a horizontal curve, inner rear wheel will be off the pavement on the inner shoulder. The off-tracking depends on
 - a) The length of the wheel base of the vehicle
 - b) The turning angle or the radius of the horizontal curve negotiated
- 2) At speeds, higher than the design speeds when the superelevation and lateral friction developed are not fully able to counteract the outwards thrust due to the centrifugal force, some transverse skidding may occur and the rear wheels may take paths on the outside of those traced by the front wheels on the horizontal curves. However, this occurs only at excessively high speeds
- 3) The path traced by the wheels of a trailer in the case of trailer units, is also likely to be on either side of the central path of the towing vehicle, depending on the speed, rigidity of the universal joints and pavement roughness
- 4) In order to take curved path with larger radius and to have greater visibility at curve, the drivers have tendency not to follow the central path of the lane, but to use the outer side at the beginning of a curve
- 5) While two vehicles cross or overtake at horizontal curve there is a psychological tendency to maintain a greater clearance between the vehicles, than on straights for increase safety

Thus, the required extra widening of the pavement at the horizontal curves, We depends on

- a) The Length of wheel based of the vehicle l ,
- b) Radius of the curve negotiated R
- c) The psychological factor which is a function of the speed of the vehicle and the radius of the curve.

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

Analysis of Extra Widening on Horizontal Curves

The extra widening of pavement on horizontal curves is divided into two parts.

1. Mechanical Widening

2. Psychological Widening

Mechanical Widening

The widening required to account for the off-tracking due to rigidity of wheel base is called as ‘Mechanical Widening’ (W_m) and is given by

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

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.

$$W_p = \frac{0.075l}{\sqrt{R}}$$

Hence Total Widening W_e is given by $W_e = W_m + W_p$

$$W_e = \frac{l^2}{2R} + \frac{0.075l}{\sqrt{R}}$$

R – Radius of the curve

n – No of lanes

l – length of wheel base of longest vehicle, m

Radius of Curve, m	Up to 20	20 to 40	41 to 60	61 to 100	101 to 300	Above 300
Extra width on two-lane pavement, m	1.5	1.5	1.2	0.9	0.6	Nil
Extra width on single lane pavement, m	0.9	0.6	0.6	Nil	Nil	Nil

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

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)

Thus, the functions of transition curve in the horizontal alignment 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 safety
- To enable gradual introduction of the designed super elevation and extra widening of pavement at the start of the circular curve.
- To improve the aesthetic appearance of the road

Type of transition curve

Different types of transition curves are

- a) Spiral or Clothoid
- b) Cubic Parabola
- c) Lemniscates

IRC recommends spiral as the transition curve because:

- 1) It fully fills the requirement of an ideal transition, as the rate of change of centrifugal acceleration is uniform throughout the length.
- 2) The geometric property of spiral is such that the calculation and setting out the curve in the field is simple and easy.

Length of transition curve

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

- 1) Rate of Change of Centrifugal Acceleration
- 2) Rate of Change of Super Elevation
- 3) An Empirical Formula Given by IRC

1. Rate of Change of Centrifugal Acceleration

At the tangent point, radius is infinity and hence centrifugal acceleration (v^2/R) is zero, as the radius is infinity. At the end of the transition, the radius R has minimum value R_m . Hence the rate of change of centrifugal acceleration is distributed over a length L_s

Let the length of transition curve be L_s m. If 't' is the time taken in seconds to traverse this transition length at uniform design speed of v m/sec, $t = L_s/v$. The maximum centrifugal acceleration of v^2/R is

introduced in time t through the transition length L_s and hence the rate of centrifugal acceleration C is given by

$$C = \frac{v^2}{Rt} = \frac{\frac{v^2}{RL_s}}{v} = \frac{v^3}{L_s R}$$

The IRC has recommended the following equation

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

The minimum and maximum value of C are limited to 0.5 and 0.8

The length of the transition curve L_s is given by

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

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The length of the transition curve L_s is given by

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

If the design speed is given in kmph

$$L_s = \frac{V^3}{46.5 CR}$$

C - rate of change of centrifugal acceleration, m/sec³

Ls – length of transition curve

R – radius of the circular curve, m

Rate of introduction of super-elevation

Raise (E) of the outer edge with respect to inner edge is given by

$$E = eB = e(W + W_e)$$

If it is assumed that the pavement is rotated about the centre line after neutralizing the camber, then the max amount by which the outer edge is to be raised at the circular curve with respect to the centre = E/2. Hence 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 Ls is given by

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

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

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

By Empirical Formula

According to IRC standards the length of horizontal transition curve Ls should not be less than the value given by the following formulas for two terrain classification

a) For plain and rolling terrain

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

b) For mountainous and steep terrain

$$L_s = \frac{V^2}{2}$$

Setting out Transition Curve

Transition curves are introduced between the tangent points of the straight stretches and the end of the circular curve on both sides. If the length of transition curve is L_s and the radius of the circular curve is R , the shift S of the transition curve is given by the formula

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

Setback Distance on Horizontal Curves

Setback distance m or the clearance distance is the distance required from the centreline 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:

- a) Required Sight Distance, S
- b) Radius of Horizontal Curve, R
- c) Length of the curve, L_c which may be greater or lesser than S

a) When $L_c > S$

When the length of curve L_c is greater than the sight distance S , let the angle subtended by the arc length S at the curve be α . On narrow roads such as single lane roads, the sight distance is measured along the centre line of the road and the angle subtended at the centre, α is equal to S/R radians. Therefore, half central angle is given by

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

The setback distance m , required from the centre line on narrow road is given by

$$m = R - R \cos \frac{\alpha}{2}$$

In case of wide roads with 2 or more lanes, if d is the distance between the centre line of the road and the centre line of the inside lane in meters, the sight distance is measured along the middle of the inner side lanes and the setback distance m' is given by

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

Where

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

b) When $L_c < S$

If the length of the curve L_c is less than the required sight distance S , then the angle α subtended at the center is determined with reference to the length of circular curve L_c and the setback distance m' is worked out in 2 parts

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

The setback distance is given by

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

Curve Resistance

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

$$CR = T - T \cos \alpha = T (1 - \cos \alpha)$$

Vertical Curves

Due to changes in grade in the vertical alignment of highway, it is necessary to introduce vertical curve at the intersections of different grades to smoothen out the vertical profile and thus ease off the changes in gradients for the fast moving vehicles. **The vertical curves used in highway may be classified into two categories:**

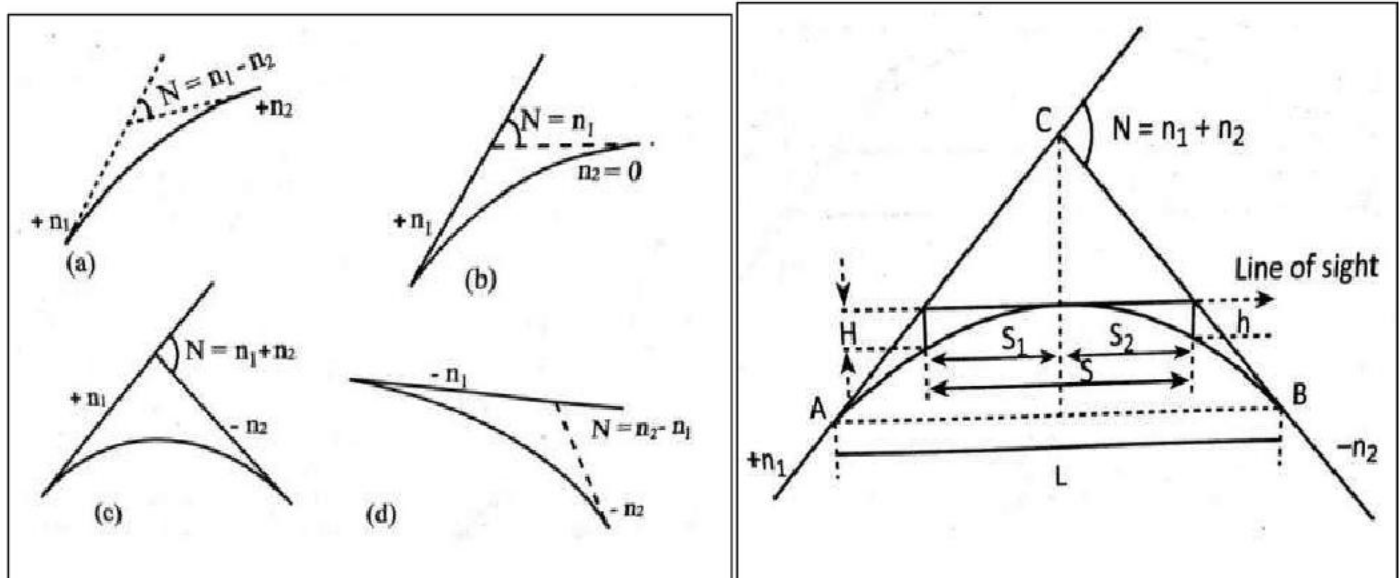
(a) **Summit curves or crest curves with convexity upwards**

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

Summit curves

Summit curves with convexity upwards are formed in any one of the cases as given below

- When a positive gradient meets another positive gradient
- When positive gradient meets a at gradient
- When an ascending gradient meets a descending gradient.
- When a descending gradient meets another descending gradien



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 is most important on summitcurves.

- 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.

DESIGN CONSIDERATION FOR LENGTH OF THE SUMMIT CURVE:

- 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 are very large, and so the a simple parabola is almost congruent to a circular arc, between the same tangent points.
- Parabolic curves is 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

Length of the summit curve for SSD

a) When $L > SSD$

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

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

Here

L – length of summit curve, m

S – SSD, m

N – Deviation angle, equal to algebraic difference in grades, radians, or tangent of deviation angle

H - Height of eye level of driver above road surface, m = 1.2m

h – Height of subject above the pavement surface, m = 0.15m

As per IRC

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

b) When L < SSD

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

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

As per IRC

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

The minimum radius of parabolic summit curve is given by R/N

Length of the summit curve for OSD or ISD

a) When L > OSD or ISD

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

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

As per IRC

$$L = \frac{NS^2}{9.6}$$

S – OSD or ISD, m

b) When $L < OSD$ or ISD

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

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

As per IRC

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

Valley curve

Valley curve or sag curves are vertical curves with convexity downwards. The deviation angle, N between the two intersecting gradients is equal to the algebraic difference between them. Among all the cases, the deviation angle will be maximum when a descending gradient, $(-n_1)$ meets with an ascending gradient, $(+n_2)$.

Therefore, deviation angle, $N = -n_1 - (+n_2) = -(n_1 + n_2)$

They are formed when two gradients meet as illustrated in figure below in any of the following four ways:

- 1) When a descending gradient meets another descending gradient
- 2) When a descending gradient meets a at gradient
- 3) When a descending gradient meets an ascending gradient
- 4) When an ascending gradient meets another ascending gradient

Length of the valley curve

The length of the valley transition curve is designed to fulfil two criteria

- a) Allowable rate change of centrifugal acceleration
- b) The required HSD for night driving

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UNIT – III TRAFFIC ENGINEERING STUDIES

TRAFFIC ENGINEERING

- A branch of Civil Engineering which deals with “*the improvement of traffic performance of road network and terminals*”
- This is achieved by systematic traffic studies, scientific analysis and engineering applications
- The method includes *planning and geometric design* on one hand and *regulation and control* on the other.

Traffic engineering therefore deals with “the application of scientific principles, tools, techniques and findings for the safe, rapid, convenient and economic movement of people and goods”

- Institute of Traffic Engineers, U.S.A defines “Traffic Engineering is that phase of engineering which deals with planning and geometric design of streets, highways, abutting lands and with traffic operation theories, as their use is related to the safe, convenient and economic transportation of persons and goods”

1.0 SCOPE OF TRAFFIC ENGINEERING

The basic objective of traffic engineering is to achieve free and rapid flow of traffic, with least number of traffic accidents.

- Factual studies of traffic operations provide the foundation for developing methods for improvement in general and for solving specific problems
- The study of traffic engineering may be divided into six major sections
 - ✓ Traffic characteristics
 - ✓ Traffic studies and analysis
 - ✓ Traffic operation, control and regulation
 - ✓ Planning and analysis
 - ✓ Geometric design
 - ✓ Administration and management

Study of traffic characteristics is the most essential prerequisites for any improvement of traffic facilities. The traffic characteristics are quite complex with various types of road users in the roads moving with different motives. The human psychology is to be given particular attention. The study of vehicular characteristics is an essential part.

The various studies to be carried out on the actual traffic include speed, volume, capacity, travel patterns, origin and destination, traffic flow characteristics, parking and accident studies.

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Various aspects covered under traffic operations are regulations, control and the warrants for application of control. Regulations may be in the form of laws and ordinances or other traffic regulatory measures such as speed limit etc. Installation of traffic control devices like signs,

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signals and islands are most common means to regulate and control the traffic. Actual adoption of traffic management measures, such as traffic regulations and control need adequate attention.

Traffic planning is separate phase for major highways like express-ways, arterial roads, mass transit facilities and parking facilities. All the aspects such as cross section and surface details, sight distance requirements, horizontal and vertical alignment, maneuverer areas and intersections and parking facilities are to be suitably designed for better performance.

The various phases of traffic engineering are implemented with the help of Engineering, Enforcement and Education (3 E's). Enforcement is usually made through traffic laws, regulations and control. Education may be possible by sufficient publicity and through schools and television. It aims at improving the human factor in traffic performance. Engineering phase is the one which is constructive. It deals with improvement of road geometrics, providing additional road facilities and installation of suitably designed traffic control devices.

1.1 TRAFFIC CHARACTERISTICS

There are five critical components that interact in a traffic system:

1. Road users - drivers, pedestrians, bicyclists, and passengers
2. Vehicles - private and commercial
3. Streets and highways
4. Traffic control devices
5. The general environment

1.1.1 Road User Characteristics

The human element is involved in all actions of the road user either as pedestrian, cyclist, cart driver or motorist. The physical, mental and emotional characteristics of human beings effect their ability to operate motor vehicle safely or to service as a pedestrian. Hence it is important to study the characteristics and limitations of the road users.

The various factors which effect road user characteristic may be broadly be classified under four heads.

1. Physical
2. Mental
3. Psychological and
4. Environmental

1.1.1.1 Physical Characteristics

The physical characteristics of the road user may be either permanent or temporary.

The permanent characteristics are the vision, hearing, strength and the general reaction to traffic situations.

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Vision plays the most important of all these. These include the acuity vision, peripheral vision and eye movement; glare vision, glare recovery and depth judgment. Field of clearest and acute vision is within a cone whose angle is only 3 degrees, though the vision is fairly satisfactory up to 10° in general and even up to 20° in the horizontal plane. However in the vertical plane the field of clear vision may be about two thirds of that in the horizontal plane. These factors are particularly taken care of while designing and installing control devices. As the field of clear vision is limited, the road user have to often shift their eyes with the peripheral field to obtain clear vision. The total time taken for the eye movement depends on some of the physical characteristics including the response to stimuli. The effect of glare adaptability to changes of light should also be studied. The depth judgment is important for a driver in judging distance and speed of vehicles and other objects ahead.

Hearing helps drivers in a way, though it is more important for pedestrians and cyclists. Though strength is not an important factor in general, lack of strength may make parking maneuvers difficult, particularly for heavy vehicles.

The reaction to traffic situation depends on the time required to perceive and understand the traffic situation and to take the appropriate action. This depends on many factors such as permanent and temporary physical factors, mental and psychological set up, speed and environmental actors. Also the time required to take appropriate action depends on the type of the problem and the familiarity.

The temporary physical characteristics of the road affecting their efficiency are fatigue, alcohol or drugs and illness. All these reduce alertness and increase the reaction time and also affect quality of judgment in some situations.

1.1.1.2 Mental Characteristics

Knowledge, skill, intelligence, experience and literacy can affect the road user characteristics. Knowledge of the vehicle characteristics, traffic behavior, driving practice, rules of road and psychology of road users will be quite useful for safe traffic operations. Reactions to certain traffic situations become more spontaneous with experience. Understanding the traffic regulation and special instruction and timely action depends on intelligence and literacy.

1.1.1.3 Psychological factors

These affect reaction to traffic situations of road users to a great extent. The emotional factors such as attentiveness, fear, anger, superstition, impatience, general attitude towards traffic and regulations and maturity also comes under this. Distractions by non-traffic events and worries reduce attentiveness to traffic situations. Dangerous actions are likely due to impatience. Some road users do not pay due regard to the traffic regulations and do not have the right attitude towards the traffic.

1.1.1.4 Environmental Factors

The various environmental conditions affecting the behavior of road users are traffic stream characteristics, facilities to the traffic, atmospheric conditions and the locality. The traffic stream may consist of mixed traffic or heavy traffic whereas the facilities to overtake

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for faster vehicles may be limited. The adaptability to different traffic stream characteristics depend on the driver's characteristics as well as the motivation. The purpose of entering the traffic stream can be social, recreational, business, routine movement or an emergency dash. The time, place and route are fundamentally chosen by the road user based on the needs. Whatever be the motive of moment, once the individual enters the traffic stream, the road user, is usually motivated by the desire for time-distance economy on one hand and comfort and safety on the other. Together with modifying factors of motivation there is a great variation among road users and their behavior in every traffic stream. The locality maybe is shopping center or a place with other distractions to the road users, thus affecting their behavior. The other environmental factors of importance are the weather visibility and other atmospheric conditions.

1.1.2 Vehicular Characteristics

It is quite important to study the various vehicular characteristics which effect the design and traffic performance because it is possible to design a road for any vehicle but not for an indefinite vehicle. The basic criterion of highway engineering is to cater for the needs of existing and anticipated traffic. It will not be economically feasible to keep on increasing the geometric standards and thickness of pavements from time to time to meet the needs of a few vehicles whose dimensions and weight are increased. The various vehicular characteristics affecting the road design may be classified as static and dynamic characteristic of the vehicle.

Static Characteristics of vehicle affecting road design are the dimensions, weight and maximum turning angle.

The height of vehicle affects the clearance of the overhead structures. The height of driver seat affects the visibility distance and the height of head light affects the headlight sight distance at valley curves. The field of vision ahead for the driver also depends on the design of wind shield and front portion of the vehicle body.

The clearance below the chassis, approach, departure and ramp angles of the vehicle affects the design of vertical profile of drive ways, humps and dips. The laden of the vehicle affects the capacity, overtaking distance and maneuverability of vehicles. The minimum turning radius depends on the length of wheel base and the features of the steering system and this effects design of sharp curves for the maneuver of vehicles at slow speeds. Gross weight, axle and wheel loads of vehicle govern the structural design of payments and cross drainage structures.

Dynamic characteristics of the vehicle affecting road design are speed, acceleration and breaking characteristics and some aspects of vehicle body design.

The speed and acceleration depends upon the power of the engine and the resistance to be overcome and are important in all the geometric design elements. The deceleration and breaking characteristics guide safe vehicle operation. The stability of vehicle and its safe movement on horizontal curves are affected by the width of wheel base and the height of center of gravity. The riding comfort on vertical curves depends on the design of suspension system of the vehicle. The impact characteristics on collision and the injuries to the occupants

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depends on the design of the bumper and body of vehicle. Some of the vehicle characteristics have been discussed below in detail.

1.1.2.1 Vehicle Dimensions

The dimensions to be mainly considered are the overall width, height and length of different vehicles, particularly of the largest ones. The width of the vehicle affects the width of the traffic lanes, shoulders and parking facilities. If the width of the lanes are not adequate in view of the widest vehicle using the roads, the capacity of road will decrease. Height of the vehicle effects the clearance to be provided under structures such as over bridges, under bridges, electric and other service lines. Length of the vehicle is an important factor in the design of horizontal alignment as it affects the extra width of pavement and minimum turning radius. Length affect the safe overtaking distance, capacity of a road and parking facilities. The length should be considered in the design of valley curves and dips. The maximum allowable width, height and length of the vehicle have been standardized by the Indian Road Congress (IRC).

Table: Maximum Dimensions of Road Vehicles

Dimension of Vehicle	Details	Maximum Dimension (m) excluding front and rear bumpers
Width	All Vehicles	2.50
Height	Single-Decked Vehicle	3.80
	Double-Decked Vehicle	4.75
Length	Single-unit truck with two or more axles	11.00
	Single-unit bus with two or more axles	12.00
	Semi-trailer tractor combinations	16.00
	Tractor and trailer combination	18.00

1.1.2.2 Weight of loaded Vehicle

The maximum weight of loaded vehicle affects the design of pavement thickness and gradients.

1.1.2.3 Power of Vehicle

The power of the heaviest vehicle and their loaded weights governs the permissible and limiting values of gradient on roads.

1.1.2.4 Speed of Vehicle

The vehicle speed affects

- Sight distance
- Superelevation, length of transition curve and limiting radius on the horizontal curves
- Length of transition curve on vertical valley curves and on humps
- Width of pavement and shoulders on straight and on horizontal curves
- Design gradient

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- Capacity of traffic lane
- Design and control measures on intersections

Thus the design speed controls most of the geometric features of highways.

1.1.2.5 Braking Characteristics

The deceleration and breaking characteristics of vehicle depend on the design and type of breaking system and its efficiency. The safety of vehicle operation, stopping distance and the spacing between the two consecutive vehicles in a traffic stream are affected by the braking capacity.

$$\text{Breaking distance } L = \frac{u^2}{2gf}$$

1.1.2.6 Off Tracking

When a four or six wheeled vehicle, such as car or bus negotiates a horizontal curve at relatively slow speed, the rear wheels do not trace the same path as the corresponding front wheels. The difference in distance between the curved wheel path of a particular set of front and rear wheels is called off tracking or the mechanical widening for a vehicle which is equal to $\frac{l^2}{2R}$

Thus the off tracking depends on two factors

- The length of wheel base or the distance between the front and rear axles of the vehicle
- The turning angle or the mean radius of the horizontal curve traversed

1.1.3 Road Characteristics

1.1.3.1 Road Surface

The type of pavement is determined by the volume and composition of traffic, the availability of materials, and available funds. Some of the factors relating to road surface like road roughness, tire wear, tractive resistance, noise, light reflection, electrostatic properties etc. should be given special attention in the design, construction and maintenance of highways for their safe and economical operation. Unfortunately, it is impossible to build road surface which will provide the best possible performance for all these conditions. For heavy traffic volumes, a smooth riding surface with good all-weather anti-skid properties is desirable. The surface should be chosen to retain these qualities so that maintenance cost and interference to traffic operations are kept to a minimum.

1.1.3.2 Lightning

Illumination is used to illuminate the physical features of the road way and to aid in the driving task. A luminaire is a complete lighting device that distributes light into patterns much as a garden hose nozzle distributes water. Proper distribution of the light flux from luminaires is one of the essential factors in efficient roadway lighting. It is important that roadway lighting be planned on the basis of many traffic information such as night vehicular traffic, pedestrian volumes and accident experience.

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1.1.3.3 Roughness

This is one of the main factors that an engineer should give importance during the design, construction, and maintenance of a highway system. Drivers tend to seek smoother surface when given a choice. On four-lane highways where the texture of the surface of the inner-lane is rougher than that of the outside lane, passing vehicles tend to return to the outside lane after execution of the passing maneuver. Shoulders or even speed-change lanes may be deliberately roughened as a means of delineation.

1.1.3.4 Pavement Colors

When the pavements are light colored (for example, cement concrete pavements) there is better visibility during day time whereas during night dark colored pavements like bituminous pavements provide more visibility. Contrasting pavements may be used to indicate preferential use of traffic lanes. A driver tends to follow the same pavement color having driven some distance on a light or dark surface, he expects to remain on a surface of that same color until he arrives a major junction point.

1.1.3.5 Night Visibility

Since most accidents occur at night because of reduced visibility, the traffic designer must strive to improve night time visibility in every way he can. An important factor is the amount of light which is reflected by the road surface to the drivers' eyes. Glare caused by the reflection of oncoming vehicles is negligible on a dry pavement but is an important factor when the pavement is wet.

1.1.3.6 Geometric aspects

The roadway elements such as pavement slope, gradient, right of way etc., affect transportation in various ways. Central portion of the pavement is slightly raised and is sloped to either sides so as to prevent the ponding of water on the road surface. This will deteriorate the riding quality since the pavement will be subjected to many failures like potholes etc. Minimum lane width should be provided to reduce the chances of accidents. Also the speed of the vehicles will be reduced and time consumed to reach the destination will also be more. Right of way width should be properly provided. If the right of way width becomes less, future expansion will become difficult and the development of that area will be adversely affected. One important other road element is the gradient. It reduces the tractive effort of large vehicles. Again the fuel consumption of the vehicles climbing a gradient is more. The other road element that cannot be avoided are curves. Near curves, chances of accidents are more. Speed of the vehicles is also affected.

1.2 Speed Studies

Speed is considered as a quality measurement of travel as the drivers and passengers will be concerned more about the speed of the journey than the design aspects of the traffic. It is defined as the rate of motion in distance per unit of time. Mathematically speed or velocity v is given by, $v = \frac{d}{t}$

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where, v is the speed of the vehicle in m/s, d is distance traveled in m in time t seconds. Speed of different vehicles will vary with respect to time and space. To represent these variation, several types of speed can be defined. Important among them are spot speed, running speed, journey speed, time mean speed and space mean speed. These are discussed below.

1.2.1 Spot Speed

Spot speed is the instantaneous speed of a vehicle at a specified location. Spot speed can be used to design the geometry of road like horizontal and vertical curves, super elevation etc. Location and size of signs, design of signals, safe speed, and speed zone determination, require the spot speed data. Accident analysis, road maintenance, and congestion are the modern fields of traffic engineer, which uses spot speed data as the basic input.

Spot speed can be measured using an enoscope, pressure contact tubes or direct timing procedure or radar speedometer or by time-lapse photographic methods. It can be determined by speeds extracted from video images by recording the distance travelling by all vehicles between a particular pair of frames.

1.2.2 Running speed

Running speed is the average speed maintained over a particular course while the vehicle is moving and is found by dividing the length of the course by the time duration the vehicle was in motion. i.e. this speed doesn't consider the time during which the vehicle is brought to a stop, or has to wait till it has a clear road ahead. The running speed will always be more than or equal to the journey speed, as delays are not considered in calculating the running speed.

1.2.3 Journey speed

Journey speed is the effective speed of the vehicle on a journey between two points and is the distance between the two points divided by the total time taken for the vehicle to complete the journey including any stopped time. If the journey speed is less than running speed, it indicates that the journey follows a stop-go condition with enforced acceleration and deceleration. The spot speed here may vary from zero to some maximum in excess of the running speed. A uniformity between journey and running speeds denotes comfortable travel conditions.

1.2.4 Time mean speed and space mean speed

Time mean speed is defined as the average speed of all the vehicles passing a point on a highway over some specified time period. Space mean speed is defined as the average speed of all the vehicles occupying a given section of a highway over some specified time period. Both mean speeds will always be different from each other except in the unlikely event that all vehicles are traveling at the same speed. Time mean speed is a point measurement while space mean speed is a measure relating to length of highway or lane, i.e. the mean speed of vehicles over a period of time at a point in space is time mean speed and the mean speed over a space at a given instant is the space mean speed.

1.2.4.1 Time mean speed (v_t)

As noted earlier, time mean speed is the average of all vehicles passing a point over a duration of time. It is the simple average of spot speed. Time mean speed v_t is given by,

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$$v_t = \frac{1}{n} \sum_{i=1}^n v_i$$

where v_i is the spot speed of i th vehicle, and n is the number of observations. In many speed studies, speeds are represented in the form of frequency table. Then the time mean speed is given by,

$$v_t = \frac{\sum_{i=1}^n q_i v_i}{\sum_{i=1}^n q_i}$$

Where q_i is the number of vehicles having speed v_i , and n is the number of such speed categories.

1.2.4.2 Space mean speed (v_s)

The space mean speed also averages the spot speed, but spatial weightage is given instead of temporal. This is derived as below. Consider unit length of a road, and let v_i is the spot speed of i th vehicle. Let t_i is the time the vehicle takes to complete unit distance and is given by $1/v_i$. If there are n such vehicles, then the average travel time t_s is given by,

$$t_s = \frac{\sum_{i=1}^n t_i}{n} = \frac{1}{n} \sum_{i=1}^n \frac{1}{v_i}$$

If t_s is the average travel time, then average speed $v_s = 1/t_s$.

Therefore, from the above equation, $v_s = \frac{n}{\sum_{i=1}^n \frac{1}{v_i}}$

If the spot speeds are expressed as a frequency table, then

$$v_s = \frac{\sum_{i=1}^n q_i v_i}{\sum_{i=1}^n \frac{q_i}{v_i}}$$

, where q_i vehicle will have v_i speed and n_i is the number of such observations.

1.2.4.3 Relation between time mean speed and space mean speed

The relation between time mean speed (v_t) and space mean speed (v_s) is given by the following relation: $v_t = v_s + \frac{\sigma^2}{v_s}$

where, σ^2 is the standard deviation of the spot speed. The standard deviation σ^2 can be computed in the following equation:

$$\sigma^2 = \frac{\sum_{i=1}^n q_i v_i^2}{\sum_{i=1}^n q_i} - (v_t)^2$$

where, q_i is the frequency of the vehicle having v_i speed.

1.2.5 Spot speed study purpose

- Establishing the speed zone of new or existing speed limit or enforcement practices.
- To determine speeds at the problem locations; to validate whether speeds are too high
- For traffic operation and control; - to establish speed limits - to determine safe speeds at curves
- Establishing speed trends at the local, state and national level to assess effectiveness of speed limit policy.

1.2.6 Factors affecting spot speed

- Driver - Age, Gender, motive of the journey, distance of his trip;
- Vehicle - type, age, weight, manufacturer and horse power;
- Roadways and environment - the graphical locations, grade, sight distance, no. of lanes, spacing of intersections; including time of day and weather
- Traffic – heavy or less volume, density, passing movements, speed regulations;

1.2.7 Locations for spot speed studies

- Speed Trend Locations: – Straight, level, open sections of rural highways – Midblock locations on urban streets
- Representative locations of different traffic conditions on a highway for basic data survey
- Problematic locations (Specific traffic engineering problem): – High accident frequency purposes – At points where the installation of traffic signals facility may be necessary

1.2.8 Methods of Measuring Spot Speeds:

The methods available for spot speed measurement may be classified as:

1. Direct observation of the time taken by a vehicle to cover a known distance.
2. Radar speedmeter method.
3. Electronic meter method
4. Photographic methods.

The direct observation method can be further classified to long-base method and short-base method.

The long-base method is sub-divided to:

- (i) Direct measurement of time
- (ii) Enoscope
- (iii) Pressure contact tubes.

1.2.8.1 Section of the Observation Point:

The criteria for this are:

1. Appropriate location for the specific purpose of study, intersection zones
2. Minimum influence of the observers and equipment on the driver and their speeds
3. Straight and level section of the highway to minimise the effect of geometric elements and roadside developments.

Selection of Base Length:

For long-base methods, the Institute of Traffic Engineers (1965) has recommended certain base lengths for spot speed determination at different average speed ranges of the traffic stream.

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Table: Basic Lengths for Spot Speed Determination:

S.No	Average Speed of Traffic Stream (kmph)	Base Length (m)
1	< 40	27
2	40 – 65	54
3	> 65	810

(i) Direct-Timing Methods:

This is the simplest and most direct method for spot speed determination. Two observers are stationed at a convenient distance (50 m) such that they can see each other. The first one starts a stop-watch as a vehicle crosses the first reference point and stops it on a signal from his counterpart the moment the vehicle touches the other end of the section.

If the timing is done by a single observer, the measurement can be made from a convenient point in the section on one side of the road at a slightly higher elevation so that both the timing spots can be seen directly.

From the known distance and the measured time intervals, speeds can be calculated. A skilled observer can read a stop-watch to an accuracy of 0.2 second. If the observer can station themselves in an inconspicuous way, the speeds calculated are not influenced by the driver's reaction to work. The disadvantages of this approach are that the readings are influenced by the observer's reaction time and the parallax effect in observing the vehicle position.

(ii) Enoscope:

The parallax error of the observer can be eliminated by using a simple device called 'Enoscope', especially in the case of a single observer. Also known as 'Mirror box', it is an L-shaped box, open at both ends; a mirror is set inside it at 45° to both the arms as shown in figure 1.1.

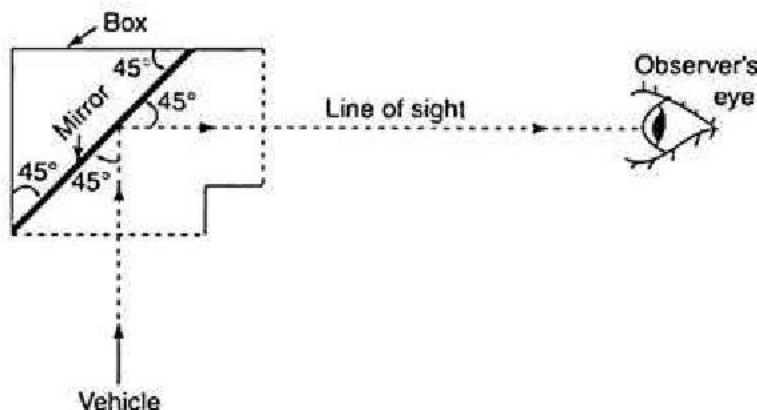


Figure 1.1 Principle of Enoscope

Light rays from the vehicle impinge on the mirror, get reflected and pass in a direction perpendicular to the incident ray. They reflected rays fall on the eye of the observer as shown. The observer can start and stop the stop-watch more accurately, with no parallax, because the line of sight is perpendicular to the direction of motion of the vehicle.

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Either one enoscope can be used or two. If only one is used the device is placed directly opposite the first reference point and the observer is stationed at the other, as shown in figure. 1.2.

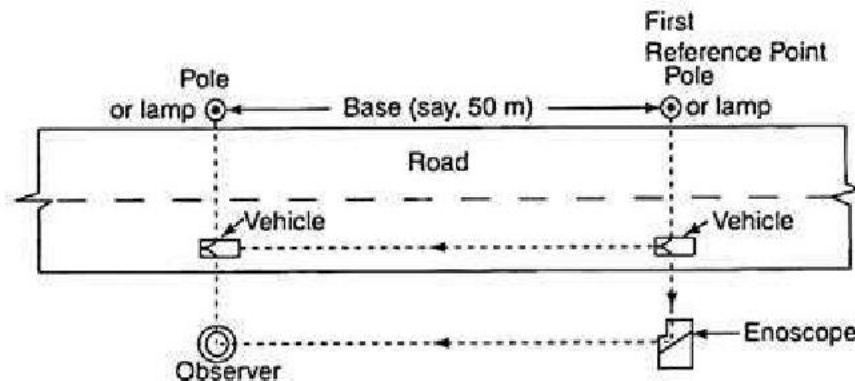


Figure 1.2 Spot Speed using Enoscope

The reference points are marked by fixing poles at the side of the road; a lamp may be used for night work. As soon as the vehicle passes the first reference point or the enoscope the timer is started and it is stopped as soon as it passes the observer. If two enoscopes are used, one each is placed at either end of the reference section and the observer will be positioned midway between the two as shown in figure 1.3.

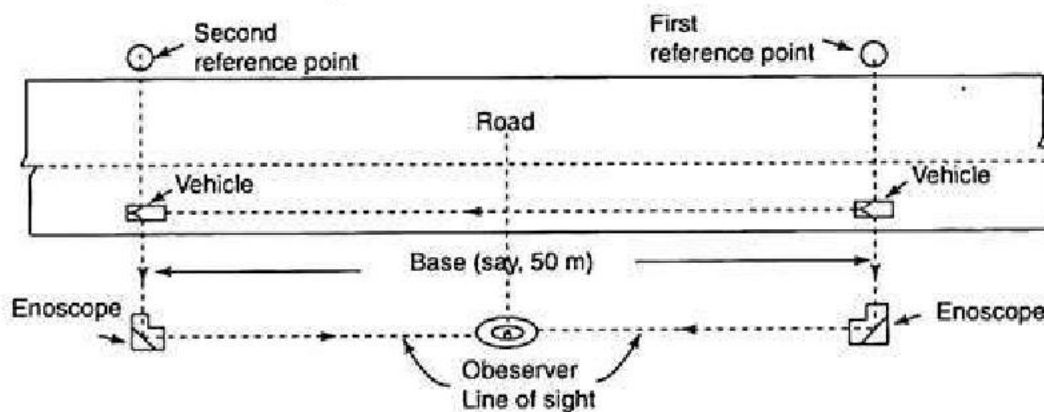


Figure 1.3: Spot Speed using two Enoscopes

The observer starts the timer when a vehicle crosses the first reference point and stops it as soon as the vehicle crosses the other end of the base by using the line of sight from each of the enoscopes. This method is not suitable for heavy multilane traffic because it is difficult to associate an observed shadow with a particular vehicle. Further, it is difficult to conceal the enoscope and the observer from the driver, whose attention would be distracted, giving scope for false readings.

(iii) Pressure Contact Tubes:

In this method, pneumatic tubes laid across the carriageway at the two reference act as detectors; when a vehicle passes over them, air impulses are sent to an electromagnetically controlled stop-watch in the hands of the observer, starting the time at the first reference point and stopping it at the second. The readings can be recorded by automatic date records also, reducing the manual work.

Short-Base Methods:

Here, the base length is very short, say 2 m to 3 m and electronic instruments are used along with pneumatic table detectors. Recording may be manual or automatic.

Radar Speedmeter Method:

The radar speedmeter works on the principle of Doppler Effect. Microwaves of radio frequency are transmitted to a moving vehicle. According to Doppler Effect, the speed of the vehicle is proportional to the change in the frequency of microwaves received back at the transmitter after being reflected by the vehicle.

The speedmeter is calibrated to read the speed directly in km/h, after the potential difference is amplified to enhance the accuracy of measurement to ± 3 km/h. The instrument is portable and battery-operated; it is set up near the edge of the road at about 1 m height above the road surface. This is used by the traffic police department for enforcing speed limits.

The operation zone can be around 50 m, and speeds can be measures for vehicles moving in both the directions.

Electronic Meter Method:

This requires the use of pneumatic tube and air switch, or electric contact stripes as detectors. On actuation, the current in milliamperes is allowed to accumulate in a condenser for a period of two electric pulses from the switches on the road. Vehicle speeds can be recorded only manually but the meter is calibrated to read the speed in km/h.

Photographic Methods:

Photography with time-lapse cameras has been successfully used to determine spot speeds of vehicles even in crowded zones. Photographs are taken at fixed intervals of time (say, one second per exposure) using a special camera. The passage of any vehicle can be traced with reference to time by projecting the exposed film on a screen. Video cameras also may be used for the purpose.

1.2.9 Presentation and Analysis of Speed Data:

Spot speed data can be analysed mathematically or graphically.

1.2.9.1 Mathematical Approach:

In this method, average speed or mean speed is obtained from a frequency distribution table by multiplying the number of vehicles in each speed class (30-40 km/h, 40 to 50 km/h, 50-65 km/h, and so on) by the mean speed of that class, summing up, and dividing the sum by the total number of vehicles observed.

Depending upon the instruments and techniques used for measuring spot speeds, the two kinds of average speeds, time-mean speed and space-mean speed, are obtained; these values give an idea of the traffic speeds in a certain stretch of the road to enable the traffic engineer to plan corrective measures, if necessary.

1.2.9.2 Graphical Approach:

The speed test data are used to form a frequency distribution table showing details of groups covering various speed ranges and number of vehicles in each range. From this table, both the histogram and the frequency distribution curve can be generated as shown in figure 1.4.

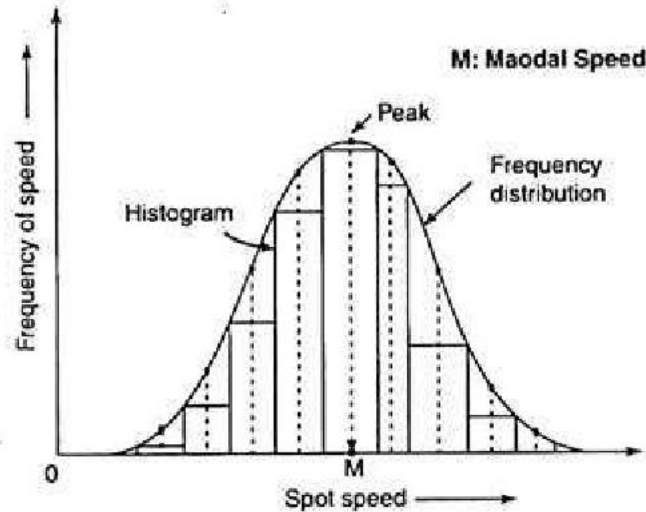


Figure 1.4: Frequency Distribution of Spot Speeds, of vehicles

For normally distributed data, the frequency curve is bell-shaped as shown. The speed corresponding to the peak is called the modal speed at which maximum number of vehicles travel.

Cumulative Spot Speeds of Vehicles:

If a graph is plotted with the average values of each speed group on the x-axis and the cumulative percentage of vehicles travelling at or below the different speeds on the y-axis, the graph appears somewhat as shown in figure. 1.5.

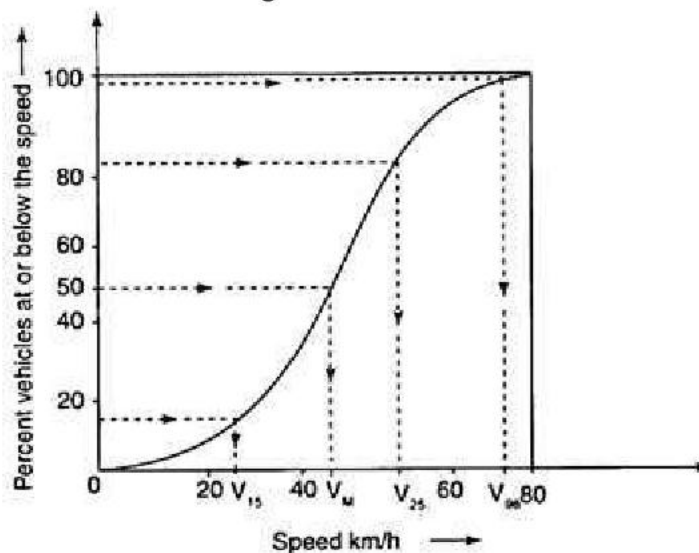


Figure 1.5: Cumulative Speed Distribution

This is very useful in determining the speed above or below which certain percentages of vehicles are moving.

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1. **Design Speed:** A speed determined for design and correlation of the physical features of highway that influence vehicle operation.
2. **85th Percentile Speed:** This is the speed below which 85% of the vehicles are moving at the point considered on the highway. In other words, only 15% of the vehicles exceed this speed at the point. This is considered to be the safe speed limit under the existing conditions in that zone. However, for the purpose of geometric design, 98 percentile speed is considered.

The speed below which 85% of all the vehicles travel is called 85th percentile speed and which is used for determining the speed limits for traffic regulation.

3. **Modal Speed:** It is the speed value occurring most frequently in a frequency distribution of speed obtained in a speed study. The modal speed would be the value with highest frequency of distribution.
4. **Median Speed:** It is the middle or 50 percentile speed. There are as many vehicles going faster than this speed as there are ones moving slower.

It is the middle value of the set of speeds when they are arranged in an ascending order or descending order. By definition, the median equally divides the distribution. Therefore, 50% of all observed speeds should be less than the median. In the cumulative frequency curve, the 50th percentile speed is the median of the speed distribution. Median Speed = v_{50}

5. **98th Percentile Speed:** It is the speed below which 98% of all the vehicles travel. It is used as design speed in geometric design.
6. **15th Percentile Speed:** This is considered to represent the minimum speed on major highways. To decrease delay and congestion and to prevent accidents, traffic which moves at a speed less than this are generally prohibited. This provides good overtaking opportunities.

It is the speed below which 15% of all the vehicles travel, is used to determine the lower speed limit on major facilities such as Expressways. Vehicles travelling below this speed cause interference with the traffic stream and may cause hazards.

1.3 Traffic Volume Studies

Systematic traffic surveys, traffic studies, and their scientific analysis are essential in traffic engineering and find wide application in planning for future needs of roads, improvement of existing facilities, geometric design aspects, pavement design, and traffic regulation and control.

Traffic surveys, also called traffic census, include types of traffic, size and weight, traffic flow, traffic volume per hour and per day, including seasonal variations/annual variations, distribution in different parts of a road network, and distribution in different directions at intersections.

1.3.1 Purposes of Traffic Volume Study

- To establish relative importance of any route or road facility.
- To decide the priority for improvement and expansion of a road and to allot the funds accordingly.
- To plan and design the existing and new facilities of traffic operations on the road.

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- To make analysis of traffic pattern and trends on the road.
- To do structural design of pavements and geometrically design of roads by classified traffic volume study.
- To plan one-way street and other regulatory measures by volume distribution study.
- To do design of road intersections, planning signal timings and channelization by turning movement study.
- To do planning of sidewalks, cross walks and pedestrian signals by pedestrian volume study.
- To do economic studies after estimating the highway user's revenue.

1.3.2 Traffic volume counts can be obtained by the following methods:

(i) Manual Method

A team of observers can record not only traffic volume but also the types of vehicles, turning movements, directions of movements, laden weights of trucks and other such details that cannot be captured by using automatic methods.

However, it is not practicable to conduct manual counts for all the 24 hours of the day and on all days of the year. But it is the most reliable method to obtain traffic volume by classified kinds of traffic and directional volume for short counts needed for intersection design. Statistical sampling techniques are used to extrapolate short-term counts.

Data may be recorded on prescribed sheets with appropriate tabular format or using multibank hand tally counters. Daily summary and weekly summary sheets are prepared.

Video Photographic Method

Video photographic method gives a permanent record of volume counts. Its analysis can be done conveniently by the use of the video which gives the required data for the analysis. By using video photographic method one can perform the classification of traffic volume easily by counting different type of vehicles. These vehicle classification counts are used in estimating structural and geometric design, computing expected highway user revenue and capacity.

However the data extraction can also be done using Traffic Data Extractor (TDE), software developed by IIT Bombay which contains tools for vehicle trajectory extraction, speed extraction and vehicle counting from a video based survey. It can be used for pedestrian data extraction also.

(ii) Automatic Counters:

Automatic counters use less number of observers and have the advantage of collecting a continuous record of traffic movement. The disadvantage is that details of classified traffic and other relevant information cannot be got.

Pneumatic Counters:

In this type, the counter is activated by an air switch attached to a flexible hose set across the road over which vehicles pass.



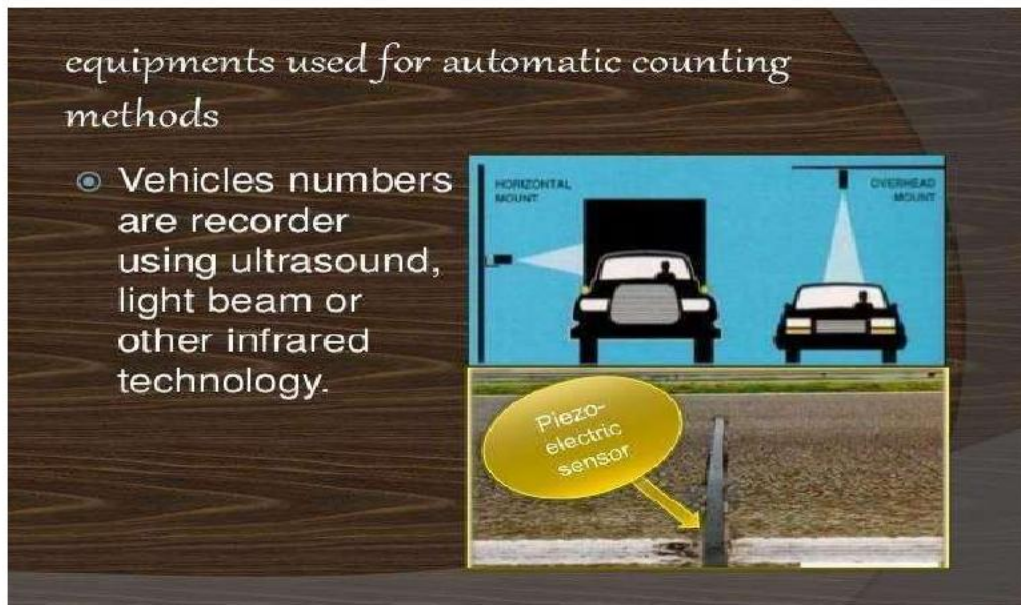
Electrical Counters:

In this method, the counter is activated by the closing of an electrical circuit by the moving vehicle.



Photo-Electric Cell Counters:

In this method, the counter is activated by the interruption of light rays falling on a photo-electric cell placed on the side of the road as the vehicle moves past it.



1.3.3 Counting Periods

The time and length that a specific location should be counted depends upon the data desired and the application in which the data are used. Counting periods vary from short counts at spot points to continuous counts at permanent stations. Hourly counts are generally significant in all engineering design, while daily and annual traffic is important in economic calculations, road system classification and investment programmes. Continuous counts are made to establish national and local highway use, trends of use and behaviour and for estimating purposes. Some of the more commonly used intervals are:

- 24-hour counts normally covering any 24-hour period between noon Monday and noon Friday. If a specific day count is desired, the count should be from midnight to midnight.
- 16 hour counts usually 5:30 am to 9:30 pm or 6 am to 9 pm.
- 12 hour counts usually from 7 am to 7 pm
- Peak Period counting times vary depending upon size of metropolitan area, proximity to major generators and the type of facility. Commonly used periods are 7 to 9 am and 4 to 6 pm.

1.3.4 Presentation of Traffic Volume Study Data:

The data obtained from traffic volume studies are presented in different forms for arriving at appropriate conclusions depending on the purpose of studies:

(a) Hourly, daily, weekly, seasonal, and annual variations of traffic in the form of charts or graphs. These will help in evaluating the existing facilities and for traffic regulation and control.

(b) Annual average daily traffic (AADT) of the total traffic as well as classified traffic are determined. The concept of 'passenger car unit' (PCU) is used to convert all types of traffic

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into an equivalent number of passenger cars. Continuous traffic counts are needed for this. Extrapolation from short counts needs the application of sampling techniques.

If the flow is measured only for short period, and not for an entire year, it is called the average daily traffic (ADT). These values help in assessing the relative importance of different routes in a road network, and also for planning improvements and phasing future development.

(c) Design hourly volume of traffic may be determined from a plot between hourly traffic volume (as percent of AADT or ADT) and the number of hours in a year when the particular traffic volume is exceeded.

(d) Traffic flow-lines are drawn along the route with the thickness of lines being proportional to the traffic volumes. This helps in knowing traffic distribution. Such lines help in fixing priorities for improvement of facilities along the routes.

(e) Traffic flow distribution at road intersections can be prepared, showing details of turning and crossing traffic volumes. This helps in the design of intersections and traffic control devices.

1.3.5 Design Hourly Volume of Traffic:

The peak hourly volume in a year will be so high that it will not be economical to design highway facilities to cater to the needs of this traffic. The average hourly volume found from annual average daily traffic (AADT) data will be inadequate for a considerable period of the year.

It has been established from considerable research that if the highway is designed for the 30th hourly traffic volume of the AADT plot, it will be satisfactory for a considerable period of the year. Since this volume will be exceeded only 29 times in the whole year, congestion will be caused only on these few occasions. This is designated as 30HV.

Thus, the thirtieth highest hourly volume is usually taken as the design hourly volume for achieving reasonable economy consistent with minimal inconvenience round the year. In some cases, this may not be strictly followed when spot study data are available and the traffic engineer uses his discretion. Sometimes, speed and origin-destination (O&D) studies may influence the decision.

Peak Flow: The peak flow is an indicative of design requirement, it is of vital concern. As used in design procedures in the United Kingdom it is usually the maximum hour's flow during a 7 day August traffic census. Since traffic is very near the year's peak in August, this maximum hour's flow is likely to approach the busiest hour of the year. However due to the influence of various factors, it is not as precise as often required.

An alternative method suggested by the Road Research Laboratory overcomes these defects by averaging the flows in a selected peak hour of each day during the highest 13 week period which, in the United Kingdom is June, July and August. This average is taken as the peak-flow hour and may be used for design purposes after suitable allowance is made for future growth.

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Typical American practice is to select one of the most highly used hours of the year, often the 30th highest, and to use the volume in this hour, after expansion, for design. By definition, the 30th highest hour is that flow which is exceeded in only 29h of the year and if the correct expansion has been made for, say, a 20 year design period, 29 h of over loading would occur a year at the end of that period.

A design hourly volume of 8 to 10 per cent of the AADT has been suggested for Indian conditions (IRC: 64-1990), based on traffic surveys on some national highways.

A typical plot from which the 30th highest hourly volume of traffic is found is shown in figure. 1.6.

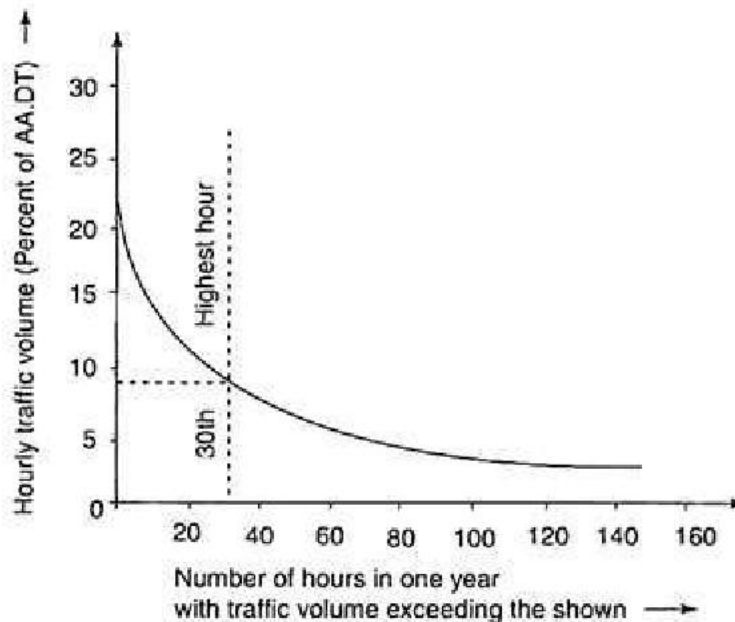


Figure 1.6 Design Hourly Traffic Volume from AADT

Peak hour factor (PHF): It is the relationship between the peak 15-minute flow rate and the peak hour volume. PHF lies between 0.25 and 1. Closer the value to unity, more uniform is the traffic and conversely closer the value to 0.25, more peaked demand will be there.

$$\text{Peak Hour Factor} = \frac{\text{Peak Hour Volume}}{\frac{60}{15} \times \text{Maximum volume at 15 min. interval in Peak Hour}}$$

Passenger Car Unit (PCU):

Highway facilities in India are used by different classes of vehicles such as cars, buses, trucks, vans, auto-rickshaws, motor-cycles, bicycles, bullock-carts and so on. The characteristics of this heterogeneous or mixed traffic flow are complex compared to homogenous traffic consisting of passenger cars only.

It is rather difficult to estimate traffic volume and capacity of road way facilities under mixed traffic flow conditions. In order to facilitate the estimation of traffic volume and traffic capacity while dealing with mixed traffic conditions, it is imperative that a common standard vehicle is chosen and all other types are converted into this class of vehicle; the standard vehicle chosen for this purpose is the passenger car. The common unit, therefore, is the passenger car unit

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(PCU). So the different vehicles classes are converted into one single class called passenger car unit (PCU).

If the addition of one particular vehicle per hour of a certain class affects the traffic flow to the same extent as the addition of x passenger cars, that particular vehicle is considered equivalent to x PCU.

The important factors that affect the PCU-value of any type of vehicle are:

1. Size – the length and width of the vehicle
2. Speed under the prevailing roadway and traffic
3. Clearances – transverse and longitudinal for ensuring safe traffic operation.
4. Regulation and control measures of traffic.
5. Road environment (urban, rural, signalized etc.)
6. Climatic and weather conditions.

Table: The IRC guidelines for capacity recommended the following PCU equivalent for different types of vehicles in India for rural conditions:

PCU-equivalents (IRC: 64–1990, Guidelines on capacity of rural roads)

S. No	Vehicle type	PCU-Equivalent factor
1	Motor cycle and scooter	0.50
2	Passenger car, van, auto rickshaw	1.00
3	Truck or bus	3.00
4	Truck-trailer / Tractor-trailer	4.50
5	Cycle	0.50
6	Cycle rickshaw	2.00
7	Handcart	3.00
8	Horse-drawn vehicle	4.00
9	Small bullock-cart	6.00
10	Big bullock-cart	8.00

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Table: The latest IRC guidelines in this regard are given below:

PCU-equivalents (IRC: 64-2009, Guidelines for capacity of roads in rural areas)

S. No	Vehicle type	PCU-Equivalent factor	
		Single and two-lane	Multi-lane
1	Passenger car, pick-up van	1.0	1.0
2	Bus, / Truck	4.0	4.5
3	Multi-axle truck	5.0	6.0
4	Light commercial vehicle	2.5	2.8
5	Tractor	3.0	3.0
6	Tractor-trailer	4.5	4.5
7	Auto-rickshaw	1.4	1.6
8	Motorised two-wheeler	0.4	0.5
9	Pedal cycle	0.6	0.6
10	Pedal rickshaw	2.0	2.5
11	Horse cart	4.0	4.0
12	Bullock cart	7.0	7.0

Table: PCU Values (IRC: 86-1983)

Sl. No	Vehicle type	PCU
1	2 wheeler	0.5
2	3 wheeler	1
3	Car/jeep	1
4	Van/minibus	1
5	LCV	1.5
6	Truck/Bus/Tractor-trailer	3
7	Bicycle	0.5

UNIT-I TRAFFIC CHARACTERISTICS

SURVEY SHEETS:

Table 2: Survey Sheet for Traffic Volume Count Data

Time Interval	2W	3W	LCV	Car	Bus	Van/Jeep	HCV	Tractor With Trailer	Bicycle	Total Volume	Total PCU
PCU	0.5	1	1.5	1	3	1	4	3	0.5		
0:0-0:15											
0:15-0:30											
0:30-0:45											
0:45-1:00											
1:0-1:15											
1:15-1:30											
1:30-1:45											
1:45-2:00											
TOTAL											

ANALYSIS:

Table 3: Traffic Volume for 1 Hour Interval

Time Interval	2W	3W	LCV	Car	Bus	Van/Jeep	HCV	Tractor	Bicycle	Total Volume	Total PCU
PCU	0.75	1.2	2	1	3	3	4	4	0.5	PCU	

OBSERVATIONS AND CALCULATIONS:

Peak hour period =

Peak 15 min period =

Maximum volume in peak 15 min period =

Maximum volume in peak hour =

$$\text{Peak Hour Factor} = \frac{\text{Peak Hour Volume}}{\frac{60}{15} \times \text{Maximum volume at 15 min. interval in Peak Hour}} =$$

1.3.6 Variation of Volume Counts and Peak Hour Factors

Variation of volume counts can be further sub-divided into daily, weekly and seasonal variation. For studying the daily variation, the flow in each hour has been expressed as percentage of daily flow. Weekdays, Saturdays and Sundays usually show different patterns. That's why comparing day with day is much more useful. Peak Hour Volume is very important factor in the design of roads and control of traffic, and is usually 2 - 2.5 times the average hourly volume. Apart from this there is one additional feature of this variation: two dominant peaks (morning and evening peak), especially in urban areas. These mainly include work trips and are not dependent on weather and other travel conditions.

Similar to daily variation, weekly variation gives volumes expressed as a percentage of total flow for the week. Weekdays flows are approximately constant but the weekend flows vary a lot depending upon the season, weather and socio-economic factors. Seasonal variation is the most consistent of all variation patterns and represents the economic and social condition of the area served.

Peak hour factors should be applied in most capacity analyses in accordance with the Highway Capacity Manual, which selected 15 minute flow rates as the basis for most of its procedures. The peak-hour factor (PHF) is descriptive of trip generation patterns and may apply to an area or portion of a street and highway system. The PHF is typically calculated from traffic counts. It is the average volume during the peak 60 minute period v_{av}^{60} divided by four times the average volume during the peak 15 minute's period v_{av}^{15} .

$$PHF = \frac{V_{av}^{60}}{4 \times V_{av}^{15}}$$

One can also use 5, 10, or 20 minutes instead of 15 minutes interval for the calculation of PHF. But in that case we have to change the multiplying factor in the denominator from 4. Generalizing,

$$PHF = \frac{V_{av}^{60}}{\frac{60}{n} \times V_{av}^n}$$

where v_{av}^n is the peak n minute flow. The Highway Capacity Manual advises that in absence of field measurements reasonable approximations for peak hour factor can be made as follows:

- 0.95 for congested condition

- 0.92 for urban areas
- 0.88 for rural areas

General guidelines for finding future PHF can be found in the Development Review Guidelines, which are as follows:

- 0.85 for minor street inflows and outflows
- 0.90 for minor arterial
- 0.95 for major streets

1.4 FUNDAMENTAL TRAFFIC FLOW PARAMETERS

The traffic stream includes a combination of driver and vehicle behavior. The driver or human behavior being non-uniform, traffic stream is also non-uniform in nature.

It is influenced not only by the individual characteristics of both vehicle and human but also by the way a group of such units interacts with each other.

Thus a flow of traffic through a street of defined characteristics will vary both by location and time corresponding to the changes in the human behavior.

Thus the traffic stream itself is having some parameters on which the characteristics can be predicted. The parameters can be mainly classified as: measurements of quantity, which includes density and flow of traffic and measurements of quality which includes speed. The traffic stream parameters can be macroscopic which characterizes the traffic as a whole or microscopic which studies the behavior of individual vehicle in the stream with respect to each other.

As far as the macroscopic characteristics are concerned, they can be grouped as measurement of quantity or quality as described above, i.e. flow, density, and speed. While the microscopic characteristics include the measures of separation, i.e. the headway or separation between vehicles which can be either time or space headway

1.4.1 Speed

Speed is considered as a quality measurement of travel as the drivers and passengers will be concerned more about the speed of the journey than the design aspects of the traffic.

It is defined as the rate of motion in distance per unit of time

$$v = \frac{d}{t}$$

where, v is the speed of the vehicle in m/s , d is distance traveled in m in time t seconds.

Speed of different vehicles will vary with respect to time and space.

1. spot speed,
2. running speed,
3. journey speed,

4. time mean speed and
5. space mean speed

Spot Speed is the instantaneous speed of a vehicle at a specified location.

- To design the geometry of road like horizontal and vertical curves, super elevation etc.
- Location and size of signs, design of signals, safe speed, and speed zone determination, require the spot speed data.
- Accident analysis, road maintenance, and congestion are the modern fields of traffic engineer, which uses spot speed data as the basic input.
- Spot speed can be measured using an enoscope, pressure contact tubes or direct timing procedure or radar speedometer or by time-lapse photographic methods.

Running speed is the average speed maintained over a particular course while the vehicle is moving and is found by dividing the length of the course by the time duration the vehicle was in motion. This speed doesn't consider the time during which the vehicle is brought to a stop, or has to wait till it has a clear road ahead. The running speed will always be more than or equal to the journey speed, as delays are not considered in calculating the running speed

Journey speed is the effective speed of the vehicle on a journey between two points and is the distance between the two points divided by the total time taken for the vehicle to complete the journey including any stopped time. If the journey speed is less than running speed, it indicates that the journey follows a stop-go condition with enforced acceleration and deceleration. A uniformity between journey and running speeds denotes comfortable travel conditions.

Time mean speed is defined as the average speed of all the vehicles passing a point on a highway over some specified time period.

Space mean speed is defined as the average speed of all the vehicles occupying a given section of a highway over some specified time period. Time mean speed is a point measurement while space mean speed is a measure relating to length of highway or lane, i.e. the mean speed of vehicles over a period of time at a point in space is time mean speed and the mean speed over a space at a given instant is the space mean speed.

1.4.2 Flow

Flow is defined as the number of vehicles that pass a point on a highway or a given lane or direction of a highway during a specific time interval. The measurement is carried out by counting the number of vehicles, n_t , passing a particular point in one lane in a defined period t . Then the flow q expressed in *vehicles/hour* is given by

$$q = \frac{n_t}{t}$$

The variation of volume with time, i.e. month to month, day to day, hour to hour and within a hour is also as important as volume calculation. Volume variations can also be observed from season to season.

The most significant variation is from hour to hour. The peak hour observed during mornings and evenings of weekdays, which is usually 8 to 10 per cent of total daily flow or 2 to 3 times the average hourly volume.

Volume in general is measured using different ways like manual counting, detector/sensor counting, moving-car observer method, etc.

Types of volume measurements

1. *Average Annual Daily Traffic (AADT)*: The average 24-hour traffic volume at a given location over a full 365-day year, i.e. the total number of vehicles passing the site in a year divided by 365.

2. *Average Annual Weekday Traffic (AAWT)*: The average 24-hour traffic volume occurring on weekdays over a full year. It is computed by dividing the total weekday traffic volume for the year by 260.

3. *Average Daily Traffic (ADT)*: An average 24-hour traffic volume at a given location for some period of time less than a year. It may be measured for six months, a season, a month, a week, or as little as two days. An ADT is a valid number only for the period over which it was measured.

4. *Average Weekday Traffic (AWT)*: An average 24-hour traffic volume occurring on weekdays for some period of time less than one year, such as for a month or a season.

1.4.3 Density

Density is defined as the number of vehicles occupying a given length of highway or lane and is generally expressed as vehicles per km. One can photograph a length of road x , count the number of vehicles, n_x , in one lane of the road at that point of time and derive the density k as,

$$k = \frac{n_x}{x}$$

1.4.4 FUNDAMENTAL RELATIONS OF TRAFFIC FLOW

The relationship between the fundamental variables of traffic flow, namely speed, volume, and density is called the fundamental relations of traffic flow. This can be derived by a simple concept. Let there be a road with length v km, and assume all the vehicles are moving with v km/hr. (figure 1.7).

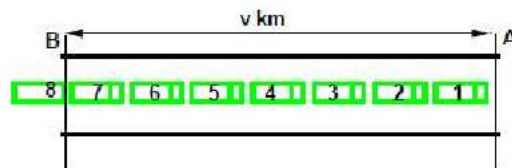


Figure 1.7: Illustration of relation between fundamental parameters of traffic flow

Let the number of vehicles counted by an observer at A for one hour be n_1 . By definition, the number of vehicles counted in one hour is flow q . Therefore,

$$n_1 = q$$

Similarly, by definition, density is the number of vehicles in unit distance. Therefore number of vehicles n_2 in a road stretch of distance v will be $density \times distance$. Therefore,

$$n_2 = k \times v$$

Since all the vehicles have speed v , the number of vehicles counted in 1 hour and the number of vehicles in the stretch of distance v will also be same.(ie $n_1 = n_2$). Therefore,

$$q = k \times v$$

This is the fundamental equation of traffic flow. Please note that, v in the above equation refers to the space mean speed.

1.4.5 Fundamental diagrams of traffic flow

The relation between flow and density, density and speed, speed and flow, can be represented with the help of some curves. They are referred to as the fundamental diagrams of traffic flow. They will be explained in detail one by one below.

1.4.5.1 Flow-density curve

The flow and density varies with time and location. The relation between the density and the corresponding flow on a given stretch of road is referred to as one of the fundamental diagram of traffic flow. Some characteristics of an ideal flow-density relationship is listed below:

1. When the density is zero, flow will also be zero, since there is no vehicles on the road.
2. When the number of vehicles gradually increases the density as well as flow increases.
3. When more and more vehicles are added, it reaches a situation where vehicles can't move. This is referred to as the jam density or the maximum density. At jam density, flow will be zero because the vehicles are not moving.
4. There will be some density between zero density and jam density, when the flow is maximum. The relationship is normally represented by a parabolic curve as shown in figure 1.8

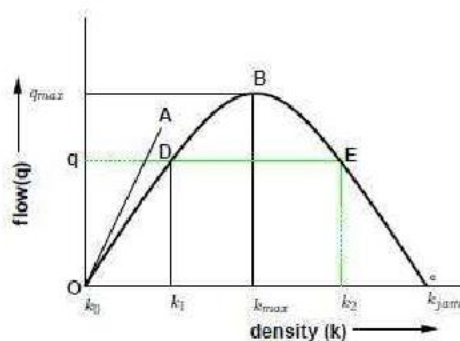


Figure1.8: Flow density curve

The point O refers to the case with zero density and zero flow. The point B refers to the maximum flow and the corresponding density is k_{max} . The point C refers to the maximum density k_{jam} and the corresponding flow is zero. OA is the tangent drawn to the parabola at O, and the slope of the line OA gives the mean free flow speed, i.e., the speed with which a vehicle

can travel when there is no flow. It can also be noted that points D and E correspond to same flow but has two different densities. Further, the slope of the line OD gives the mean speed at density k_1 and slope of the line OE will give mean speed at density k_2 . Clearly the speed at density k_1 will be higher since there are less number of vehicles on the road.

1.4.5.2 Speed-density diagram

Similar to the flow-density relationship, speed will be maximum, referred to as the free flow speed, and when the density is maximum, the speed will be zero. The simplest assumption is that this variation of speed with density is linear as shown by the solid line in figure 1.9. Corresponding to the zero density, vehicles will be moving with their desire speed, or free flow speed. When the density is jam density, the speed of the vehicles becomes zero.

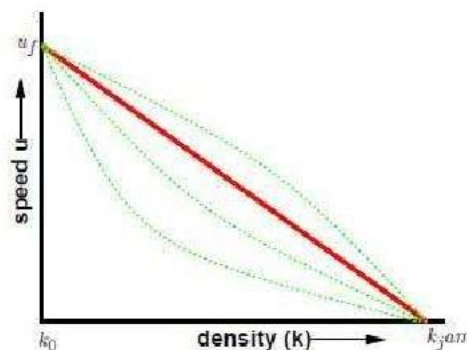


Figure 1.9: Speed-density diagram

1.4.5.3 Speed flow relation

The relationship between the speed and flow can be postulated as follows. The flow is zero either because there is no vehicles or there are too many vehicles so that they cannot move. At maximum flow, the speed will be in between zero and free flow speed. This relationship is shown in figure 1.10. The maximum flow q_{max} occurs at speed u . It is possible to have two different speeds for a given flow.

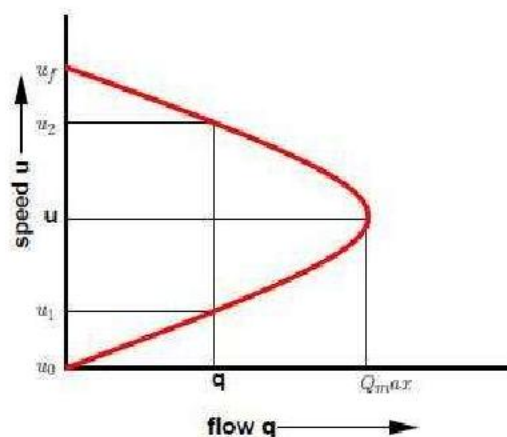


Figure 1.10: Speed-flow diagram

1.4.5.4 Combined diagrams

The diagrams shown in the relationship between speed-flow, speed-density, and flow-density are called the fundamental diagrams of traffic flow. These are as shown in figure 1.11

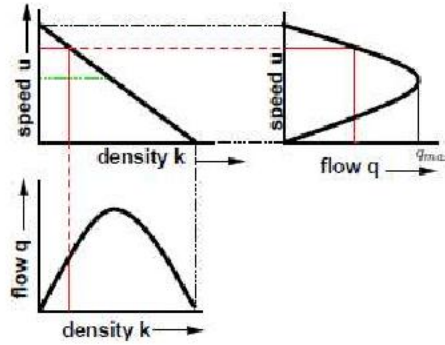


Figure 1.11: Fundamental diagram of traffic flow

- A linear speed-density relationship is illustrated in figure 1.9.

$$v \rightarrow v_f \text{ when } k \rightarrow 0$$

$$k \rightarrow k_j \text{ when } v \rightarrow 0$$

- The equation for this relationship is shown below.

$$v = v_f - \left[\frac{v_f}{k_j} \right] \cdot k$$

Where v is the mean speed at density k , v_f is the free speed and k_j is the jam density.

- we know that $q = kv$

$$\text{Therefore } q = v_f \cdot k - \left[\frac{v_f}{k_j} \right] \cdot k^2$$

- To find density at maximum flow, differentiate equation with respect to k and equate it to zero.

$$\begin{aligned} \frac{dq}{dk} &= 0 \\ v_f - \frac{v_f}{k_j} \cdot 2k &= 0 \\ k &= \frac{k_j}{2} \end{aligned}$$

- Denoting the density corresponding to maximum flow as k_0 ,

$$k_0 = \frac{k_j}{2}$$

- Therefore, density corresponding to maximum flow is half the jam density. Once we get k_0 , we can derive for maximum flow, q_{max}

$$\begin{aligned} q_{max} &= v_f \cdot \frac{k_j}{2} - \frac{v_f}{k_j} \cdot \left[\frac{k_j}{2} \right]^2 \\ &= \frac{v_f k_j}{4} \end{aligned}$$

- Thus the maximum flow is one fourth the product of free flow and jam density.
- The speed at maximum flow, v_0 ,

$$v_0 = v_f - \frac{v_f}{k_j} \cdot \frac{k_j}{2}$$

$$v_o = \frac{v_f}{2}$$

- Therefore, speed at maximum flow is half of the free speed.

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Numerical Examples

1. If the spot speeds are 50, 40, 60, 54 and 45, then find the time mean speed and space mean speed.

Solution:

Time mean speed v_t is the average of spot speed.

$$\text{Therefore } v_t = \frac{\sum v_i}{n} = \frac{50+40+60+54+45}{5} = \frac{249}{5} = 49.8$$

Space mean speed is the harmonic mean of spot speed.

$$\text{Therefore } v_s = \frac{n}{\sum \frac{1}{v_i}} = \frac{5}{\frac{1}{50} + \frac{1}{40} + \frac{1}{60} + \frac{1}{54} + \frac{1}{45}} = \frac{5}{0.12} = 48.82$$

2. The results of a speed study is given in the form of a frequency distribution table. Find the time mean speed and space mean speed.

Speed Range	Frequency
2 – 5	1
6 – 9	4
10 – 13	0
14 - 17	7

Solution:

If the spot speeds are expressed as a frequency table, then

$$\text{Time mean speed } v_t = \frac{\sum_{i=1}^n q_i v_i}{\sum_{i=1}^n q_i}$$

$$\text{Space mean speed } v_s = \frac{\sum_{i=1}^n q_i}{\sum_{i=1}^n \frac{q_i}{v_i}}$$

S.No	Speed Range	Average speed (v_i)	Frequency (q_i)	$v_i q_i$	$\frac{q_i}{v_i}$
1	2 – 5	3.5	1	3.5	0.28
2	6 – 9	7.5	4	30.0	0.54
3	10 – 13	11.5	0	0	0
4	14 - 17	15.5	7	108.5	0.45
Total			12	142	1.27

$$\text{Time mean speed } v_t = \frac{\sum_{i=1}^n q_i v_i}{\sum_{i=1}^n q_i} = \frac{142}{12} = 11.83$$

$$\text{Space mean speed } v_s = \frac{\sum_{i=1}^n q_i}{\sum_{i=1}^n \frac{q_i}{v_i}} = \frac{12}{1.27} = 9.45$$

3. For the data given below, compute the time mean speed and space mean speed. Finally compute the density of the stream.

Speed Range	Frequency
0 – 10	5
10 – 20	15
20 – 30	20
30 – 40	25
40 – 50	30

Solution:

If the spot speeds are expressed as a frequency table, then

$$\text{Time mean speed } v_t = \frac{\sum_{i=1}^n q_i v_i}{\sum_{i=1}^n q_i}$$

$$\text{Space mean speed } v_s = \frac{\sum_{i=1}^n q_i}{\sum_{i=1}^n \frac{q_i}{v_i}}$$

S.No	Speed Range	Average speed (v_i)	Frequency (q_i)	$v_i q_i$	$\frac{q_i}{v_i}$
1					
2					
3					
4					
Total					

$$\text{Time mean speed } v_t = \frac{\sum_{i=1}^n q_i v_i}{\sum_{i=1}^n q_i} = - =$$

$$\text{Space mean speed } v_s = \frac{\sum_{i=1}^n q_i}{\sum_{i=1}^n \frac{q_i}{v_i}} = - =$$

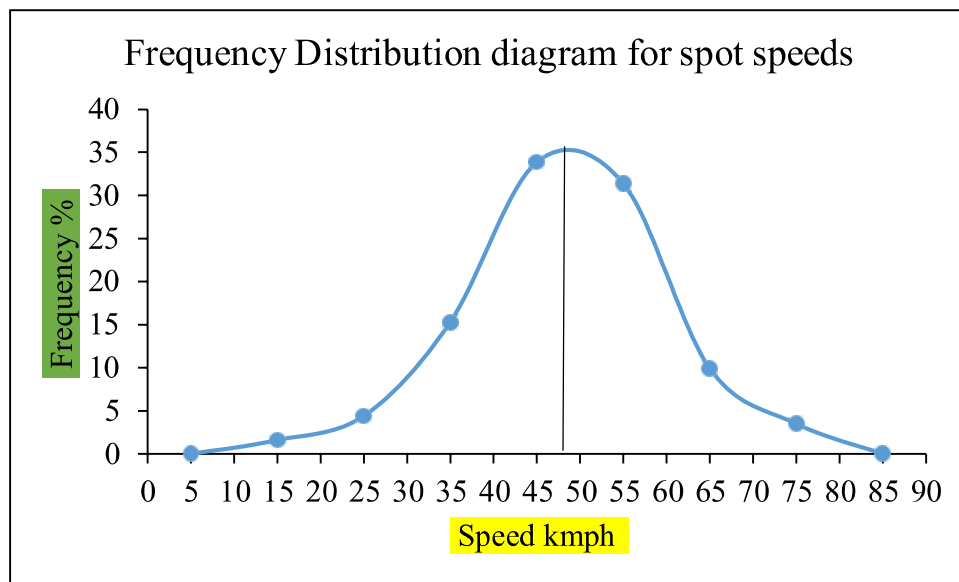
4. The table below gives the consolidated data of spot speed studies on a section of a road. Determine the most preferred speed at which maximum proportion of vehicles travel

Speed Range (kmph)	Frequency
0 – 10	0
10 – 20	11
20 – 30	30
30 – 40	105
40 – 50	233
50 – 60	216

60 – 70	68
70 – 80	24
80 – 90	0

Solution:

Speed Range (kmph)	Mean Speed (kmph)	Frequency	Percent frequency
A	B	C	D
0 – 10	5	0	0.0
10 – 20	15	11	1.6
20 – 30	25	30	4.4
30 – 40	35	105	15.3
40 – 50	45	233	33.9
50 – 60	55	216	31.4
60 – 70	65	68	9.9
70 – 80	75	24	3.5
80 – 90	85	0	0.0



The most preferred speed is the modal speed. It corresponds to the peak value and is found as 47kmph

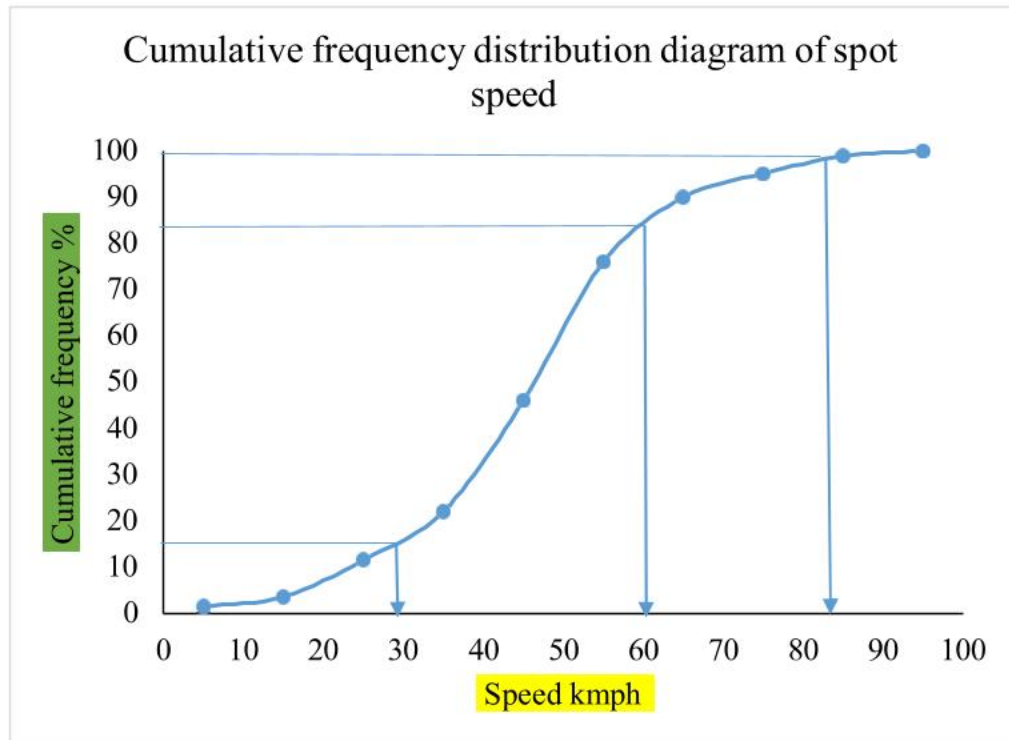
- Spot speed studies were carried out at a certain stretch of a highway with mixed traffic flow and the consolidated data collected are given below.

Speed range, kmph	No of vehicles observed
0 - 10	12
10 - 20	18
20 - 30	68
30 - 40	89
40 - 50	204
50 - 60	255
60 - 70	119
70 - 80	43
80 - 90	33
90 - 100	9

Determine: (i) The upper and lower values or speed limits for installing speed regulation signs at this road stretch and (ii) the design speed for checking the geometric design elements of the highway.

Solution:

Speed range, kmph	Mean Speed kmph	Frequency	% Frequency	Cumulative Frequency, (at or below the speed), %
0 - 10	5	12	1.41	1.41
10 - 20	15	18	2.12	3.53
20 - 30	25	68	8	11.53
30 - 40	35	89	10.47	22
40 - 50	45	204	24	46
50 - 60	55	255	30	76
60 - 70	65	119	14	90
70 - 80	75	43	5.06	95.06
80 - 90	85	33	3.88	98.94
90 - 100	95	9	1.06	100
Total		850	100	



- (a) Upper speed limit for regulation = 85th percentile speed = 60 kmph
 - (b) Lower speed limit for regulation = 15th percentile speed = 30 kmph
 - (c) Speed to check geometric design elements = 98th percentile speed = 84 kmph
6. The free mean speed on a roadway is found to be 80kmph. Under stopped condition the average spacing between vehicles is 6.9m. Determine the capacity flow.

Solution:

Given Free mean speed $V_{sf} = 80\text{kmph}$

$$s_{jam} = 6.9\text{m}$$

$$\text{Jam density } k_j = \frac{1000}{6.9} = 145 \text{ veh/km}$$

$$\text{Maximum flow } q_{max} = \frac{V_{sf} K_j}{4} = \frac{80 \times 145}{4} = 2900 \frac{\text{veh}}{\text{hr}} / \text{lane}$$

7. The table below shows the volumetric data observed at an intersection. Calculate the peak hour volume, peak hour factor (PHF), and the actual (design) flow rate for this approach

Time Interval	Cars
4:00 - 4:15	30
4:15 - 4:30	26
4:30 - 4:45	35
4:45 - 5:00	40

5:00 - 5:15	49
5:15 - 5:30	55
5:30 - 5:45	65
5:45 - 6:00	50
6:00 - 6:15	39
6:15 - 6:30	30

Solution:

- The peak hour volume is just the sum of the volumes of the four 15 minute intervals within the peak hour (219). The peak 15 minute volume is 65 in this case. The peak hour factor (PHF) is found by dividing the peak hour volume by four times the peak 15 minute volume. $PHF = \frac{219}{4 \times 65} = 0.84$
- The actual (design) flow rate can be calculated by dividing the peak hour volume by the PHF, $219/0.84 = 260$ vehicles/hr, or by multiplying the peak 15 minute volume by four, $4 \times 65 = 260$ vehicles per hour.

8. Find the peak hour factor for the 15-minute vehicle volumes given

- 1) V15 = 190
- 2) V30 = 200
- 3) V45 = 190
- 4) V60 = 190

Solution:

$$PHF = (190 + 200 + 190 + 190) / [(200 \times 4)] = 0.96$$

9. Find the peak hour factor for the 15-minute vehicle volumes given

- 1) V15 = 175
- 2) V30 = 200
- 3) V45 = 150
- 4) V60 = 125

Solution:

$$PHF = (175 + 200 + 150 + 125) / [(200 \times 4)] = 0.81$$

A PHF of 1 indicates that the traffic volume in every 15-minute interval is the same and therefore the traffic flow is consistent throughout the hour. Lower PHF values indicate more variable traffic flows and that the traffic volume has a spike during the peak 15-minute interval.

10. The table below shows flow rates that were measured for four 15-minute time periods for each of the 12 intersection movements. Find the peak hour factor

Time period	Eastbound			Westbound			Northbound			Southbound			Total
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
4:00 PM	40	55	175	50	50	75	120	815	45	40	700	55	2,220
4:15 PM	50	75	375	55	80	125	215	1,025	20	60	1,975	165	4,220

4:30 PM	30	75	125	45	75	115	20	975	35	55	1,200	145	2,895
4:45 PM	45	60	175	55	85	150	145	1,015	45	50	1,350	130	3,305

Solution:

Examination of this table shows that second time period, which begins at 4:15 pm, is the peak 15-minute period of the four that are shown here.

The total flow for this time period (15 min) is 4,220 veh,

The average flow rate for the hour is 12,640 veh/hr; this is the sum of the total volumes observed during each of the four 15-minute periods shown below.

The peak hour factor can then be computed as follows:

$$\begin{aligned}
 \text{Peak Hour Factor} &= \frac{\text{Peak Hour Volume}}{\frac{60}{15} * \text{Maximum volume at 15 min. interval in Peak Hour}} \\
 &= 12,640 / 16,880 \\
 &= 0.75
 \end{aligned}$$

11. The Flow in each 5-minute interval from 4:30 – 6:00Pm is given in the table below. Determine when the peak hour begins and ends. What is the peak hour factor?

Time	Flow Rate
4:30 - 4:35	1200
4:35 - 4:40	1350
4:40 - 4:45	1490
4:45 - 4:50	1780
4:50 - 4:55	1720
4:55 - 5:00	1540
5:00 - 5:05	1670
5:05 - 5:10	1650
5:10 - 5:15	1700
5:15 - 5:20	1400
5:20 - 5:25	1500
5:25 - 5:30	1700
5:30 - 5:35	1300
5:35 - 5:40	1300
5:40 - 5:45	1550
5:45 - 5:50	1350
5:50 - 5:55	1780
5:55 - 6:00	1450

Solution:

Table: Flow in 15minute interval

Time	Flow Rate	15 Min Interval	15 Min flow Rate
4:30 - 4:35	1200	4:30 – 4:45	
4:35 - 4:40	1350		
4:40 - 4:45	1490		
4:45 - 4:50	1780	4:45 – 5:00	
4:50 - 4:55	1720		
4:55 - 5:00	1540		
5:00 - 5:05	1670	5:00 – 5:15	
5:05 - 5:10	1650		
5:10 - 5:15	1700		
5:15 - 5:20	1400	5:15 – 5:30	
5:20 - 5:25	1500		
5:25 - 5:30	1700		
5:30 - 5:35	1300	5:30 – 5:45	
5:35 - 5:40	1300		
5:40 - 5:45	1550		
5:45 - 5:50	1350	5:45 – 6:00	
5:50 - 5:55	1780		
5:55 - 6:00	1450		

Table: Flow in 1 Hour interval

1 Hour interval	1 Hour Volume
4:30 – 5:30	
4:45 – 5:45	
5:00 – 6:00	

Peak hour period =

Peak 15 min period =

Maximum volume in peak 15 min period =

Maximum volume in peak hour =

Peak Hour Factor = $\frac{\text{Peak Hour Volume}}{\frac{60}{15} \times \text{Maximum volume at 15 min. interval in Peak Hour}}$ =

12. The table below shows the volumetric data collected at an intersection: Calculate the peak hour volume, peak hour factor (PHF), and the actual (design) flow rate for this approach.

Table: Volumetric Data Collected

Time	HCV	LCV	CAR	3W	2W
4:00 - 4:15	4	10	6	38	24
4:15 - 4:30	8	12	9	63	33
4:30 - 4:45	7	13	8	42	27
4:45 - 5:00	6	13	15	37	32

5:00 - 5:15	7	14	10	51	28
5:15 - 5:30	6	10	9	63	41
5:30 - 5:45	8	11	8	48	38
5:45 - 6:00	10	6	15	47	21
6:00 - 6:15	9	7	9	54	26
6:15 - 6:30	10	9	11	62	35
6:30 - 6:45	12	11	12	61	39
6:45 - 7:00	8	8	10	54	42

Table: PCU Values

HCV	3.5
LCV	2.2
CAR	1.0
3W	0.8
2W	0.5

Solution:

The first step in this solution is to find the total traffic volume for each 15 minute period in terms of passenger car units. For this purpose the PCU values given in the table are used. Once we have this, we can locate the hour with the highest volume and the 15 minute interval with the highest volume.

Time	HCV	LCV	CAR	3W	2W	Flow in PCU
PCU	3.5	2.2	1.0	0.8	0.5	
4:00 - 4:15						
4:15 - 4:30						
4:30 - 4:45						
4:45 - 5:00						
5:00 - 5:15						
5:15 - 5:30						
5:30 - 5:45						
5:45 - 6:00						
6:00 - 6:15						
6:15 - 6:30						
6:30 - 6:45						
6:45 - 7:00						

Table: Flow in 1 Hour interval

1 Hour interval	1 Hour Volume
4:00 – 5:00	
4:15 – 5:15	
4:30 – 5:30	
4:45 – 5:45	
5:00 – 6:00	
5:15 – 6:15	
5:30 – 6:30	
5:45 – 6:45	

6:00 – 7:00	
-------------	--

Peak hour period =

Peak 15 min period =

Maximum volume in peak 15 min period =

Maximum volume in peak hour =

Peak Hour Factor = $\frac{\text{Peak Hour Volume}}{\frac{60}{15} \times \text{Maximum volume at 15 min. interval in Peak Hour}}$ =

The actual (design) flow rate can be calculated by dividing the peak hour volume by the PHF, or by multiplying the peak 15 minute volume by 4.

Therefore

The actual design flow rate = peak hour volume/PHF

Or

The actual design flow rate = peak 15min volume*4

13. Determine the time mean speed, space mean speed, and 85th percentile speed from the following speed (in m/s) data

Speed	Frequency
1- 5	9
6-10	16
11-15	32
16-20	48
21-25	23
26-30	9

14. For a given road following speed data is collected. 25, 31, 36, 39, 42, 44, 47, 48, 49, 51, 52, 52, 53, 54, 55, 56, 57, 57, 57, 58, 59, 60, 60, 62, 63, 64, 65, 66, 66, 68, 68, 69, 70, 70, 71, 73, 75, 79, 85, 89, 90. What is the speed you will recommend for designing sight distance or radius of circular curve?
15. The spot speeds of ten vehicles observed at a certain location are 55.1, 40.8, 32.2, 47.8, 64.5, 53.2, 58.2, 67.6, 36.4, and 53.2 kmph. Find the time mean speed, space mean speed and 85th percentile speed
16. Table below indicates the values of spot speeds observed at a certain location. Find (i) the 98th percentile speed (ii) 85th percentile speed (iii) 50th percentile speed and (iv) 15th percentile speed

Obs. No	Speed (kmph)
1	55.1
2	40.8

3	32.2
4	47.8
5	64.5
6	53.2
7	58.2
8	66.6
9	36.4
10	53.2

HIGHWAY CAPACITY AND LEVEL OF SERVICE

2.0 General

From the fundamental traffic flow characteristics like flow, density, and speed, a few other parameters of traffic flow can be derived. Significant among them are the time headway, distance headway and travel time. They are discussed one by one below.

2.1 Derived Characteristics

2.1.1 Time headway

The microscopic characteristic related to volume is the time headway or simply headway. Time headway is defined as the time difference between any two successive vehicles when they cross a given point. Practically, it involves the measurement of time between the passage of one rear bumper and the next past a given point. If all headways h in time period t , over which flow has been measured are added then,

$$\sum_{1}^{n_t} h_i = t$$

But the flow is defined as the number of vehicles n_t measured in time interval t , that is,

$$q = \frac{n_t}{t} = \frac{n_t}{\sum_{1}^{n_t} h_i} = \frac{1}{h_{av}}$$

where, h_{av} is the average headway. Thus average headway is the inverse of flow. Time headway is often referred to as simply the headway.

2.1.2 Distance headway

Another related parameter is the distance headway. It is defined as the distance between corresponding points of two successive vehicles at any given time. It involves the measurement from a photograph, the distance from rear bumper of lead vehicle to rear bumper of following vehicle at a point of time. If all the space headways in distance x over which the density has been measured are added,

$$\sum_{1}^{n_x} s_i = x$$

But the density (k) is the number of vehicles n_x at a distance of x , that is

$$k = \frac{n_x}{x} = \frac{n_x}{\sum_{1}^{n_x} s_i} = \frac{1}{s_{av}}$$

Where, s_{av} is average distance headway. The average distance headway is the inverse of density and is sometimes called as spacing.

HIGHWAY CAPACITY AND LEVEL OF SERVICE

2.1.3 Travel time

Travel time is defined as the time taken to complete a journey. As the speed increases, travel time required to reach the destination also decreases and vice versa. Thus travel time is inversely proportional to the speed. However, in practice, the speed of a vehicle fluctuates over time and the travel time represents an average measure.

2.2 Gap acceptance and follow-up time

Gap acceptance is one of the most important components in microscopic traffic characteristic. The gap acceptance theory commonly used in the analysis of uncontrolled intersections based on the concept of defining the extent drivers will be able to utilize a gap of particular size or duration. A driver entering into or going across a traffic stream must evaluate the space between a potentially conflicting vehicle and decide whether to cross or enter or not. One of the most important aspects of traffic operation is the interaction of vehicles with in a single stream of traffic or the interaction of two separate traffic streams. This interaction takes place when a driver changes lanes merging in to a traffic stream or crosses a traffic stream. Inherent in the traffic interaction associated with these basic maneuvers is concept of gap acceptance.

2.2.1 Basic Terminologies

1. **Gap** means the time and space that a subject vehicle needs to merge adequately safely between two vehicles. Gap acceptance is the minimum gap required to finish lane changing safely. Therefore, a gap acceptance model can help describe how a driver judges whether to accept or not.
2. **Gap acceptance:** The process by which a minor stream vehicle accepts an available gap to maneuver.
3. **Critical gap:** The minimum major-stream headway during which a minor-street vehicle can make a maneuver.
4. **Lag:** Time interval between the arrival of a yielding vehicle and the passage of the next priority stream vehicle (Forward waiting time).
5. **Headway:** The time interval between the arrivals of two successive vehicles. Headway differs from gap because it is measured from the front bumper of the front vehicle to the front bumper of the next vehicle.
6. **Minimum Headway:** The minimum gap maintained by a vehicle in the major traffic stream.
7. **Follow-up time:** Time between the departure of one vehicle from the minor street and the departure of the next vehicle using the same gap under a condition of continuous queuing.
8. **Delay:** The additional travel time experienced by a driver, passenger or pedestrian.
9. **Conflicting movements:** The traffic streams in conflict at an intersection.
10. **Capacity:** The maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or a roadway during a given time period under prevailing roadway, traffic, and control conditions.

HIGHWAY CAPACITY AND LEVEL OF SERVICE

2.2.2 Critical Gap

The critical gap t_{cx} for movement x is defined as the minimum average acceptable gap that allows intersection entry for one minor street or major street. The term average acceptable means that the average driver would accept or choose to utilize a gap of this size. The gap is measured as the clear time in the traffic stream defined by all conflicting movements. Thus, the model assumes that all gaps shorter than t_{cx} are rejected or unused, while all gaps equal to or larger than t_{cx} would be accepted or used. The adjusted critical gap t_{cx} computed as follows.

$$t_{cx} = t_{cb} + t_{cHV}P_{HV} + t_GG + t_c + t_{3LT}$$

where, t_{cx} is the critical gap for movement x , t_{cb} is the base critical gap from Table. 2.1 t_{cHV} is the adjustment factor for heavy vehicles P_{HV} is the proportion of heavy vehicles t_{cG} is the adjustment factor for grade G is the percent grade divided by 100, t_{cT} is the adjustment factor for each part of a two-stage gap acceptance process, and t_{3LT} is the critical gap adjustment factor for intersection geometry.

2.2.3 Follow-up time

The follow up time t_{fx} for movement x is the minimum average acceptable time for a second queued minor street vehicle to use a gap large enough admit two or more vehicles. Follow-up times were measured directly by observing traffic flow. Resulting follow-up times were analyzed to determine their dependence on different parameters such as intersection layout. This measurement is similar to the saturation flow rate at signalized intersection. Table 2.1 and 2.2 shows base or unadjusted values of the critical gap and follow up time for various movements. Base critical gaps and follow up times can be adjusted to account for a number of conditions, including heavy - vehicle presence grade, and the existence of two stage gap acceptance. Adjusted Follow up Time computed as:

$$t_{fx} = t_{fb} + t_{fHV}P_{HV}$$

where, t_{fx} is the follow-up time for minor movement x , t_{fb} is the base follow-up time from table 2.2, t_{fHV} is the adjustment factor for heavy vehicles, and P_{HV} is the proportion of heavy vehicles for minor movement.

HIGHWAY CAPACITY AND LEVEL OF SERVICE

Table 2.1 Base critical gap and follow up times

Vehicle Movement	Base Critical Gap, $t_{c,base}$ (s)		Base Follow-up
	Two-Lane Major Street	Four-Lane Major Street	Time $t_{f,base}$ (s)
Left turn from major	4.1	4.1	2.2
Right turn from minor	6.2	6.9	3.3
Through traffic on minor	6.5	6.5	4.0
Left turn from minor	7.1	7.5	3.5

Table 2.2 Adjustments to base critical gap and follow up times

Adjustment Factor	Values(s)	
t_{cHV}	1.0	Two-lane major streets
	2.0	Four-lane major streets
t_{cG}	0.1	Movements 9 and 12
	0.2	Movements 7,8,10 and 11
	1.0	Otherwise
t_{cT}	1.0	First or second stage of two-stage process
	0.0	For one-stage process
T_{3LT}	0.7	Minor-street LT at T-intersection
	0.0	Otherwise
t_{fHV}	0.9	Two-lane major streets
	1.0	Four-lane major streets

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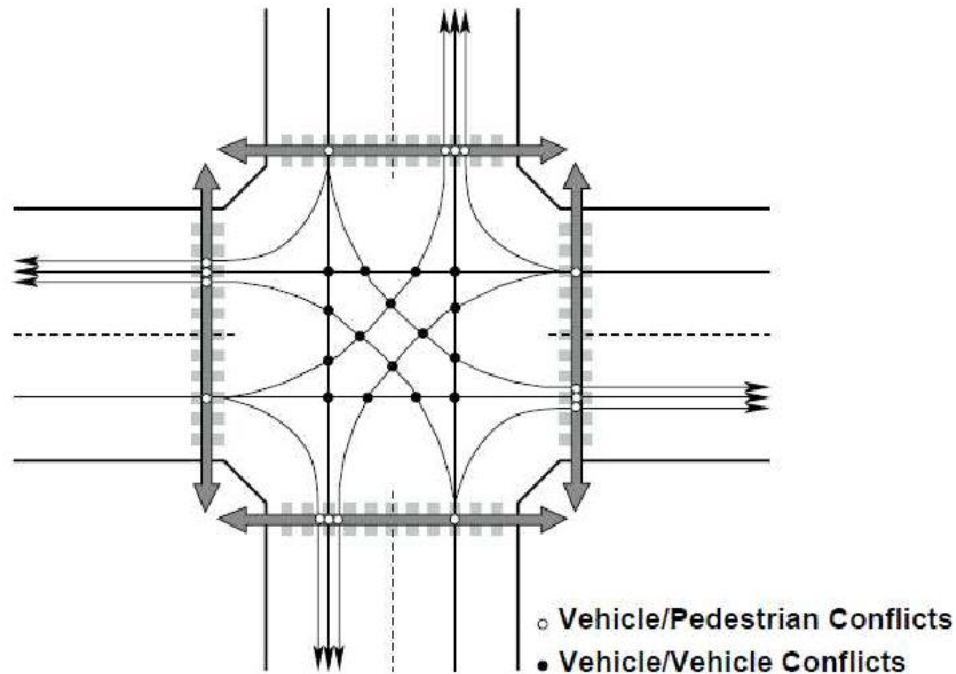


Figure 2.1 Conflicts at four legged intersection

2.2.4 Conflicting volume

The traffic flow process at un-controlled intersection is complicated since there are many distinct vehicular movements to be accounted for. Most of this movements conflict with opposing vehicular volumes. These conflicts result in decreasing capacity, increasing delay, and increasing potentials for traffic accidents. Consider a typical four-legged intersection as shown in Fig. 2.1. The numbers of conflicts for competing through movements are 4, while competing right turn and through movements are 8. The conflicts between right turn traffics are 4, and between left turn and merging traffic are 4. The conflicts created by pedestrians will be 8 taking into account all the four approaches. Diverging traffic also produces about 4 conflicts. Therefore, a typical four legged intersection has about 32 different types of conflicts. Conflicts at an intersection are different for different types of intersection. The essence of the intersection control is to resolve these conflicts at the intersection for the safe and efficient movement of both vehicular traffic and pedestrians.

2.3 Capacity and Level of Service LOS

Often it is required to ascertain how much a transport facility can accommodate. Such information is useful in the design of traffic facility. Capacity analysis helps in answering the question. It is a quantitative assessment of the ability of a traffic facility to handle vehicles or people for which it is designed.

A related question is, what is the performance level of the system at various operating conditions. Or in other words, how good is the operation of the traffic facility. Level of Service analysis tries

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to answer this question which is essentially a qualitative analysis. Capacities and Level of Services are therefore closely related analysis of a traffic facility.

Capacity and Level of service are two related terms. Capacity analysis tries to give a clear understanding of how much traffic a given transportation facility can accommodate. Level of service tries to answer how good is the present traffic situation on a given facility. Thus it gives a qualitative measure of traffic, whereas capacity analysis gives a quantitative measure of a facility. Capacity and level of service varies with the type of facility, prevailing traffic and road conditions etc.

Importance of the Concept of Highway Capacity

The concept of highway capacity is important for the following reasons:

1. The capacity of a highway should be adequate to serve the needs of the projected traffic.
2. The class of highway, lane width, number of lanes and intersections are dependent on capacity.
3. Improvements on geometric elements, traffic control devices and traffic management measures can be effectively planned based on the studies of highway capacity.
4. The adequacy of the existing highway network for the existing traffic volume can be assessed by capacity studies; transportation planning can be done effectively using this information.

2.3.1 Capacity

Capacity of a transport facility is defined as the maximum number of vehicles, passengers, or the like, per unit time which can be accommodated under given conditions with a reasonable expectation of occurrence. The Highway Capacity Manual (HCM 2010) defines the capacity as the maximum hourly rate at which persons or vehicles can be reasonably expected to traverse a point or a uniform segment of a lane or roadway during a given time period, under prevailing roadway, traffic and control conditions.

Capacity is defined as the maximum number of vehicles, passengers, or the like, per unit time, which can be accommodated under given conditions with a reasonable expectation of occurrence. Some of the observations that are found from this definition can be now discussed. Capacity is independent of the demand. It speaks about the physical amount of vehicles and passengers a road can afford. It does not depend on the total number of vehicles demanding service. On the other hand, it depends on traffic conditions, geometric design of the road etc. For example, a curved road has lesser capacity compared to a straight road. Capacity is expressed in terms of units of some specific thing (car, people, etc.), so it also does depend on the traffic composition. In addition, the capacity analysis depends on the environmental conditions too. Capacity is a probabilistic measure and it varies with respect to time and position. Hence it is not always possible to completely derive analytically the capacity. In most cases it is obtained, through field observations

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Basic capacity is the maximum number of passenger cars that can pass a given point on a Lane or roadway during one hour under nearly ideal roadway and traffic conditions which can possibly be attained. Two roads having the same physical features will have the same basic capacity irrespective of traffic conditions, as they are assumed to have ideal conditions. Thus basic capacity is the *theoretical capacity*.

Possible capacity is the maximum number of vehicles that can pass a given point on a Lane or roadway during one hour under prevailing roadway and traffic conditions. The possible capacity of a road is generally much lower than the basic capacity as the prevailing roadway and traffic conditions are seldom ideal. In a worst case when the prevailing traffic condition is so bad that due to traffic condition, the traffic may come to a stand-still condition and in such a situation the possible capacity of the road may approach zero.

When the prevailing roadway and traffic conditions approach the ideal conditions, the possible capacity would also approach the basic capacity. Thus the value of possible capacity varies from zero to basic capacity. For the purpose of design neither basic capacity nor possible capacity can be adopted as they represent two extreme cases of roadway and traffic conditions.

Practical capacity is the maximum number of vehicles that can pass a given point on a Lane or roadway during one hour without traffic density being so high as to cause unreasonable delay, hazard or restriction to the driver's freedom to manoeuvre under the prevailing roadway and traffic conditions. It is the practical capacity which is of primary interest to the designers who strive to provide adequate highway facilities and hence this is also called *design capacity*.

The capacity measure depends on these operating conditions. The first is the traffic conditions and the factors that influence the capacity includes vehicle composition, turning, movements, etc. The second factor is the roadway conditions and it includes geometrical characteristics such as lane width, shoulder width, horizontal alignment, vertical alignment. The third factor is the control conditions such as the traffic signal timings, round-about characteristics.

Determination of theoretical maximum capacity

An estimate of a theoretical maximum or basic capacity of a single Lane may be made from the relation:

$$= \frac{1000V}{S}$$

Here,

q_c = capacity of a single lane, vehicles per hour per lane

V = speed, Kmph

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S = average centre to centre spacing of vehicles when they follow one behind the other as a queue or space headway, m

Thus the capacity depends upon the speed V and spacing S . The average spacing between centre to centre vehicles is equal to the average length of vehicle plus the average clear gap between the vehicles in the stream and this spacing is also equal to the space headway. The minimum clear gap between vehicles are allowed for safe stopping of the rear vehicle in case the vehicle ahead suddenly stops. It is also found that drivers follow the vehicle ahead at a closer gap at a lower speeds and the clear gap and the spacing H_s gets increased instinctively at higher speed of the traffic stream.

Thus the space gap allowed by the driver of a following vehicle depends on several factors such as,

- a) Speed of leading and following vehicles
- b) Type and characteristics of the two vehicles
- c) Driver characteristics of the following vehicle
- d) Traffic volume to capacity ratio of the road stretch at the instant or the 'level of service'
- e) The proportion of different vehicle classes in the stream
- f) Roadway geometrics and
- g) Environmental factors such as weather conditions, lighting, etc.

The assumption that space gap increases in direct proportion to the speed of the vehicle or that of the traffic stream gives only an approximate average value of the space gap between vehicles in the traffic stream. The space gap between the vehicle ahead and the following vehicle in a traffic stream is sometimes assumed to be equal to the distance travelled during the reaction time of the driver, with a further assumption that the breaking distance of the lead vehicle and the following vehicle is approximately equal. If the reaction time is t sec, the minimum space gap S_g is given by

$$S_g = vt, m = 0.278Vt, m$$

The minimum space headway, S in a traffic stream is therefore equal to the minimum space gap plus average length of vehicle L in the stream.

$$S = S_g + L = 0.278Vt + L$$

In a stream flow, as the driver of the following vehicle is quite alert, the average reaction time is found to be low; this value is often assumed to be 0.70 to 0.75 sec. In the analysis of overtaking sight distance, the value of reaction time has been assumed as 0.7 sec in the empirical relation for spacing, S given by,

$$S = (0.7V + L) = (0.2V + L), m$$

Thus a suitable value of spacing S maybe adopted in above equation to estimate the theoretical capacity of a traffic Lane with the homogeneous traffic flow.

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It has been observed (as explained earlier) that with increase in speed of the traffic stream the time headway decreases and after reaching a minimum value at an optimum speed, starts increasing. The maximum theoretical capacity of a traffic lane may therefore be obtained if the minimum time headway H_t is known.

$$q_c = \frac{3600}{H_t}$$

Here q_c is the capacity, vehicles per hour (or 3600sec), and the minimum time headway in seconds.

The peak value of theoretical capacity is reached at an optimum speed. As the speed is increase further, the maximum capacity flow of the lane starts decreasing due to increase in headway at the higher speed range.

Factors affecting practical capacity

Some of the important factors that affect the practical capacity of a traffic lane or is Highway are listed below:

- a) *Lane width*: As the Lane width decreases, the capacity also decreases. The practical capacity of 3.0m wide Lane or a two-lane road in rural areas may decrease to 76% of the capacity of a 3.5m lane.
- b) *lateral clearance*: vertical obstructions such as retaining walls, raised culvert walls or Parked vehicles near the traffic lane reduces the effective width of a lane and thus result in a reduction in the capacity of the adjacent lane. Further restricted lateral clearance affects driving comfort and increases accident rate. As minimum clearance of 1.85m from the pavement edge to the obstruction is considered desirable so that capacity is not affected adversely. When the distance from payment edge to an obstruction decreases to 0.75m on one side only, the capacity decreases to 96% and when this obstruction is on both sides, the percentage further decreases to 80% of standard design capacity.
- c) *Width of shoulders*: Narrow shoulders reduce the effective width of adjacent traffic lanes as a vehicles travel towards the center of the payment. When vehicles in emergency (like that of a tyre puncture or other break down) have to park on the shoulders of insufficient width, there is a reduction in effective width of the adjacent lane, resulting in a considerable reduction in the capacity of the lane.
- d) *Commercial vehicles*: Large commercial vehicles like a truck and buses occupy greater space and influence the other traffic in the same lane as well as the vehicles along the adjoining lanes. Also these heavy commercial vehicles may travel at lower speeds especially on ascending grades.
- e) *Alignment*: If the alignment and geometrics are not up to their desired standards, the capacity will decrease. Particularly restrictions to sight distance requirements cause reduction in capacity. Steep and long grades affect the capacity. When 60% of the road

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length has sub-standard OSD, the capacity decreases to 65% of the standard design capacity.

- f) *Road Geometrics*: Presence of sharp horizontal curves, steep gradients, summit and Valley curves necessitate speed changes and consequently result in lower highway capacity.
- g) *Presence of intersections at grade*: Intersections restrict free flow of traffic and thus adversely affect the capacity. The capacity of an intersection of two roads crossing at grade will be slightly less than the road with the lower capacity of the two. At signalized intersections as the vehicles have to stop alternately to allow crossing traffic, The capacity of the intersections will be further decreased. In order to provide consistent traffic flow and maximum capacity on important highways, is necessary to plan them as controlled access highways with grade separated intersections.
- h) Other factors which affect the capacity are the stream speed, one or two way traffic movement, number of traffic lanes, vehicular and driver characteristics, composition of traffic and the traffic volume.

2.3.2 Level of service

Level-of-Service (LOS) of a traffic facility is a concept introduced to relate the quality of traffic service to a given flow rate. Level-of-Service is introduced by HCM to denote the level of quality one can derive from a local under different operation characteristics and traffic volume. HCM proposes LOS as a letter that designate a range of operating conditions on a particular type of facility. Six LOS letters are defined by HCM, namely A, B, C, D, E, and F, where A denote the best quality of service and F denote the worst. These definitions are based on Measures of Effectiveness (MoE) of that facility. Typical measure of effectiveness include speed, travel-time, density, delay etc. There will be an associated service volume for each of the LOS levels. A service volume or service flow rate is the maximum number of vehicles, passengers, or the like, which can be accommodated by a given facility or system under given conditions at a given LOS.

2.3.3 Type of Facilities

HCM has developed the capacities standard and LOS measure for various facilities. Each traffic facility has its own unit for the capacity and measure of effectiveness for each item will also vary. The traffic facilities can be divided into three, namely: the uninterrupted facilities, interrupted facilities, and others. Uninterrupted facilities include freeway (basic freeway, weaving sections, and ramps), multi-lane highways (unidirectional), two-lane highways (bidirectional). Freeways normally have density as the measure of effectiveness, while multi-lane and two-lane highways have delay/speed as the MoE. Interrupted facilities include un-signalized intersection, signalized intersection, and arterials or corridors. They have respectively control delay, total delay and average travel speed as the measure of effectiveness. Other facilities may include pedestrian pathways, bicycle tracks, bus-transit system, rail-transit system and air-transportation system. Each of them have facility specific measure of effectiveness.

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Factors affecting level of service

The level of service can be derived from a road under different operating characteristics and traffic volumes. The factors affecting level of service (LOS) can be listed as follows:

- a) Speed and travel time
- b) Traffic interruptions/restrictions
- c) Freedom to travel with desired speed
- d) Driver comfort and convenience
- e) Operating cost.

3 Illustrations

For a typical freeway mid-block section the capacity and LOS can be defined for an ideal section. An ideal section has uninterrupted flow from both sides and has only passenger cars and the drivers are regular travelers who are familiar with the facility. The lane width is 3.65m wide with proper shoulder and 1.8m lateral clearance is available from the edge of the pavement. The free flow speed of 115kmph is achievable on the multi-lane and 100kmph on the two-lane highway.

3.1 Capacity

Such a facility is considered as an ideal facility and for such facilities the following values can be taken as capacity.

- a) A capacity of 2000 vehicle per hour per lane for a speed of 115kmph
- b) A capacity of 1900 vehicles per hour per lane for a speed of 80kmph
- c) A capacity of 2800 vehicle per hour for both direction at 100kmph

Note that the above values are not analytical or experimentally derived, but, statistically derived from the observed field values from large number of such sections. Needly to say that it is possible to have a flow higher than this capacity measure, but not necessary.

3.2 Level of service

The above capacity value drop due to various 'non-ideal condition' which includes changes in speed or travel time, traffic interruptions or restriction etc. Accordingly HCM has defined various levels of services for the traffic facility. Assigning quality value is based on several user surveys capturing the perception of drivers on the quality of the traffic under various operating condition. The Figure 2.2 illustrate the quality of services or Level-of-Services (A to F) and the various operating conditions. The same can be shown in the Table 2.3.

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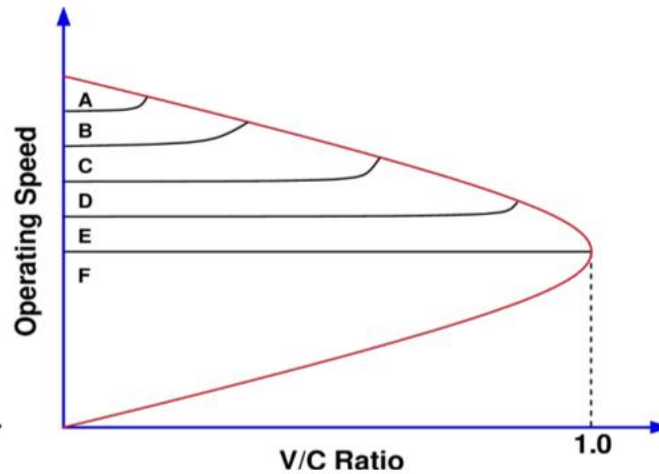


Figure 2.2 The Level of Service of a mid-block section is expressed in terms of the operating speed and volume to capacity (v/c) ratio

Table 2.3 LOS of a Mid-Block Section

LOS	Quality	Speed (kmph)	V/C	Description
<i>A</i>	Free-flow	80	0.6	High level of physical and psychological comfort
<i>B</i>	Reasonable free-flow	70	0.7	Reasonable level of physical and psychological comfort
<i>C</i>	Near free-flow	60	0.8	Local deterioration possible with blockages
<i>D</i>	Medium flow	50	0.85	Non-recoverable local disruptions
<i>E</i>	At capacity flow	40	0.9	Minor disturbances resulting breakdown
<i>F</i>	Congested flow	15	1	Break down of flow capacity drops

Level of service A represents the zone of free flow. Here the traffic volume will be less, traffic will be experiencing free flow also. The drivers will be having the complete freedom to choose their desired speed. Even at maximum density, for this LOS the average spacing between vehicles is 167 m. Lane changes within the traffic stream, as well as merging and diverging movements, are made relatively easy. The effect of minor incidents and point breakdowns are easily aborted at this level. Level of service B represents zone of reasonably free flow. Free flow speeds are still maintained at this level of service. The drivers freedom to choose their desired speed is only slightly restricted. The lowest average spacing between vehicles is about 100 m. The effects of small incidents and point breakdowns are still easily contained. At level of service C, the presence of other vehicles begins to restrict the maneuverability within the traffic stream. Average speeds

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remain at or near the free flow speed level, but significant increase in driver vigilance is required at this level. Minimum average spacing between the vehicles is in the range of 67 m. Queues may be expected to form behind any significant blockage. At level of service D, the average speeds begin to decline with increasing flows. Freedom to maneuver within the traffic stream is noticeably restricted. At this level, density deteriorates more quickly with flow. The spacing between the vehicles is about 50 m. As the traffic stream has little space to absorb disruptions, minor incidents can lead to queuing of vehicles. Level of service E define operation at capacity. At this level, the stream reaches its maximum density limit. There will be no usable gaps in the stream and even slight disruptions will cause a breakdown, with queues forming rapidly behind the disruption. Maneuvering within the traffic stream becomes extremely difficult. Level of service F describes conditions in a queue that has formed behind a point of breakdown or disruption. As vehicles shuffle through the queue, there may be periods when they move quickly, and others when they are stopped completely. Thus this level of service is used to describe the point of breakdown as well, even though operations downstream of such a breakdown may appear good. Level of service F represents the region of forced flow, having low speed, and complete breakdown of the system.

2.4 Multilane Highways

2.4.1 Introduction

Increasing traffic flow has forced engineers to increase the number of lanes of highways in order to provide good maneuvering facilities to the users. The main objectives is to analyze LOS which is very important factor for a traffic engineer because it describes the traffic operational conditions within a traffic stream. Also we are going to study the characteristics and capacity for multilane highways. Free-flow speed is an important parameter that is being used extensively for capacity and level-of- service analysis of various types of highway facilities.

2.4.2 Multilane Highways

A highway is a public road especially a major road connecting two or more destinations. A highway with at least two lanes for the exclusive use of traffic in each direction, with no control or partial control of access, but that may have periodic interruptions to flow at signalized intersections not closer than 3.0 km is called as multilane highway. They are typically located in suburban areas leading to central cities or along high-volume rural corridors that connect two cities or important activity centers that generate a considerable number of daily trips.

2.4.2.1 Highway Classification

There are various ways of classification of highways; we will see classification of highways according to number of lanes.

- Two lane highways.
- Multilane highways

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2.4.2.2 Highway Characteristics

Multilane highways generally have posted speed limits between 60 km/h and 90 km/h. They usually have four or six lanes, often with physical medians or two-way right turn lanes (TWRTL), although they may also be undivided. The traffic volumes generally varies from 15,000 - 40,000 vehicles per day. It may also go up to 100,000 vehicles per day with grade separations and no cross-median access. Traffic signals at major intersections are possible for multilane highways which facilitate partial control of access. Typical illustrations of multilane highway configurations are provided in Figure 2.3 and 2.4

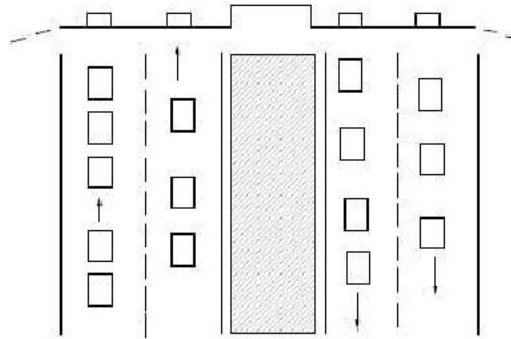


Figure 2.3 Divided multilane highway in a rural/suburban environment

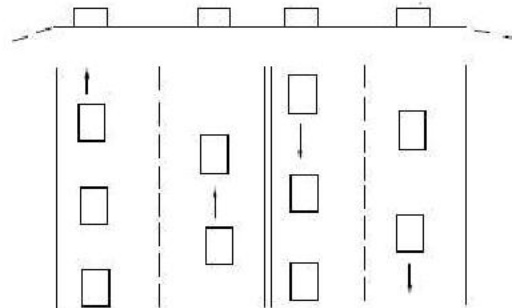


Figure 2.4 Undivided multilane highway in a rural/suburban environment

2.4.3 Highway Capacity

An important operation characteristic of any transport facility including the multilane highways is the concept of capacity. Capacity may be defined as the maximum sustainable flow rate at which vehicles or persons reasonably can be expected to traverse a point or uniform segment of a lane or roadway during a specified time period under given roadway, traffic, environmental, and control conditions; usually expressed as vehicles per hour, passenger cars per hour, or persons per hour. There are two types of capacity, possible capacity and practical capacity. Possible capacity is defined as the maximum number of vehicles that can pass a point in one hour under prevailing roadway and traffic condition. Practical capacity on the other hand is the maximum number that can pass the point without unreasonable delay restriction to the average driver's freedom to pass

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other vehicles. Procedure for computing practical capacity for the uninterrupted flow condition is as follows:

1. Select an operating speed which is acceptable for the class of highways the terrain and the driver.
2. Determine the appropriate capacity for ideal conditions from table 2.4.
3. Determine the reduction factor for conditions which reduce capacity (such as width of road, alignment, sight distance, heavy vehicle adjustment factor).
4. Multiply these factors by ideal capacity value obtained from step 2.

Table 2.4 Free flow speed and capacity for multilane highway

Types of facility	Free flow speed(kmph)	Capacity (pcphpl)
Multilane	100	2200
	90	2100
	80	2000
	70	1900

BASE CONDITIONS FOR CAPACITY ESTIMATION (Multilane divided interurban highways as per INDO-HCM 2017)

The base conditions for capacity estimation of multilane divided interurban highways are given below:

- Plain and level terrain
- No horizontal curvature or vertical gradient
- Traffic lanes 3.5 m wide [As per IRC: SP-84 (2014) and IRC: SP-87 (2013), the standard lane width of highways in India should be 3.5 m.]
- Minimum 1.5 m wide paved shoulders, followed by unpaved shoulders/crash barriers, the latter necessarily required in case of sections on an embankment of more than 3 m height
- Minimum of 2.5 m wide median
- No pavement defects that can adversely affect traffic operations. Roughness, in terms of International Roughness Index (IRI), is less than 2.7 m/km.
- No direct access points
- No side friction in terms of public transport stops, bus bays and truck lay-byes coupled with absence of significant pedestrian activity or slow-moving vehicles such as bicycles, rickshaws and animal carts.
- No work zone activity at or near the section.
- No incidents or crashes at the time of observation at or near the section.

BASE CONDITIONS FOR CAPACITY ESTIMATION (Two lane as per INDO-HCM 2017)

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The basic parameters of traffic flow, namely speed, volume and density are used to estimate the capacity of road. For the identification of base section, predefined conditions followed for bidirectional carriageways covering varying road typologies are as follows:

- The carriageway width should be 7.0 m with a minimum of 1.0 m soft shoulder to facilitate two-way traffic movement in the case of two lane bidirectional roads. Similarly, in the case of intermediate lane road, being a road with restricted carriageway width, the carriageway width should be 5.5 m with a minimum of 1.0 m soft shoulder to facilitate two-way traffic movement at reduced speeds. In the case of single lane roads, the carriageway width should be 3.75 m with soft shoulder of 1.0 m.
- The section should be straight and level.
- The section should not be influenced by interruptions such as intersections, steep gradients and curvatures as well as any other adjoining roads.
- There must not be any physical barrier on at least 500 m section such as speed breakers, rumble strips, as it may affect the traffic stream.
- Section must be free from any form of roadside friction activities.
- Section should be free from any form of work activity for at least 1 km on either side.
- No incidents or crashes at the time of observation for at least 1 km on either side.

2.4.4 Level of Service

Level of service (LOS) is a qualitative term describing the operational performance of any transportation facility. The qualitative performance measure can be defined using various quantitative terms like:

1. Volume to capacity ratio,
2. Mean passenger car speed,(in km/h)
3. Density, (in p/kmln).

Basically any two of the following three performance characteristics can describe the LOS for a multilane highway. Each of these measures can indicate how well the highway accommodates the traffic demand. Since speed does not vary over a wide range of flows, it is not a good indicator of service quality. Density which is a measure of proximity of other vehicles in the traffic stream and is directly perceived by drivers and does not vary with all flow levels and therefore density is the most important performance measure for estimating LOS. Based on the quantitative parameter, the LOS of a facility can be divided into six qualitative categories, designated as LOS A,B,C,D,E,F. The definition of each level of service, is given below:

Level of Service A

Travel conditions are completely free flow. The only constraint on the operation of vehicles lies in the geometric features of the roadway and individual driver preferences. Lane changing, merging and diverging manoeuvre within the traffic stream is good, and minor disruptions to traffic

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are easily absorbed without an effect on travel speed. Average spacing between vehicles is a minimum of 150 m or 24 car lengths. Figure 2.5 shows LOS A.



Figure 2.5 LOS A

Level of Service B

Travel conditions are at free flow. The presence of other vehicles is noticed but it is not a constraint on the operation of vehicles as are the geometric features of the roadway and individual driver preferences. Minor disruptions are easily absorbed, although localized reduction in LOS are noted. Average spacing between vehicles is a minimum of 150 m or 24 car lengths. Figure 2.6 below shows LOS B.



Figure 2.6 LOS B

Level of Service C

Traffic density begins to influence operations. The ability to manoeuvre within the traffic stream is affected by other vehicles. Travel speeds show some reduction when free-flow speeds exceed 80 km/h. Minor disruptions may be expected to cause serious local deterioration in service, and queues may begin to form. Average spacing between vehicles is a minimum of 150m or 24 car length. Figure 2.7 shows LOS C.

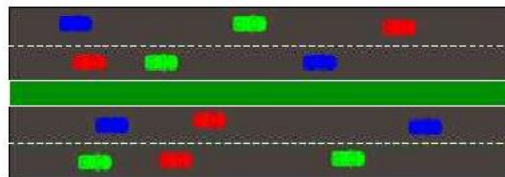


Figure 2.7 LOS C

Level of Service D

The ability to manoeuvre is severely restricted due to congestion. Travel speeds are reduced as volumes increase. Minor disruptions may be expected to cause serious local deterioration in

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service, and queues may begin to form. Average spacing between vehicles is a minimum of 150m or 24 car length. Figure 2.8 shows LOS D.

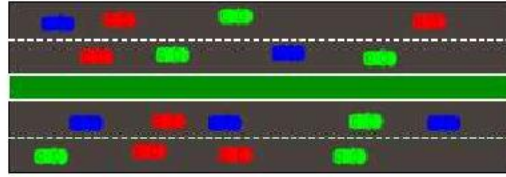


Figure 2.8 LOS D

Level of Service E

Operations are unstable at or near capacity. Densities vary, depending on the free-flow speed. Vehicles operate at the minimum spacing for which uniform flow can be maintained. Disruptions cannot be easily dissipated and usually result in the formation of queues and the deterioration of service to LOS F. For the majority of multilane highways with free-flow speed between 70 and 100km/h, passenger-car mean speeds at capacity range from 68 to 88 km/h but are highly variable and unpredictable. Average spacing between vehicles is a minimum of 150 m or 24 car length. Figure 2.9 shows LOS E.

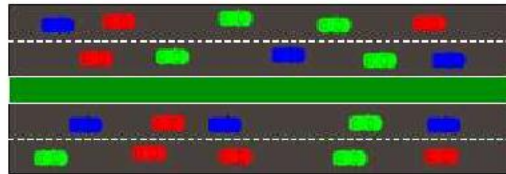


Figure 2.9 LOS E

Level of Service F

A forced breakdown of flow occurs at the point where the numbers of vehicles that arrive at a point exceed the number of vehicles discharged or when forecast demand exceeds capacity. Queues form at the breakdown point, while at sections downstream they may appear to be at capacity. Operations are highly unstable, with vehicles experiencing brief periods of movement followed by stoppages. Travel speeds within queues are generally less than 48 km/h. Note that the term LOS F may be used to characterize both the point of the breakdown and the operating condition within the queue. Figure 2.10 shows LOS F.

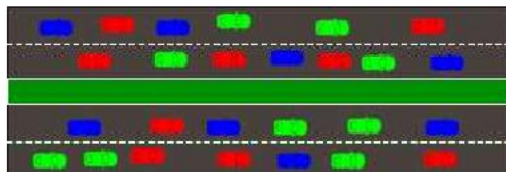


Figure 2.10 LOS F

2.5 Determination of Level of Service

The determination of level of service for a multilane highway involves three steps:

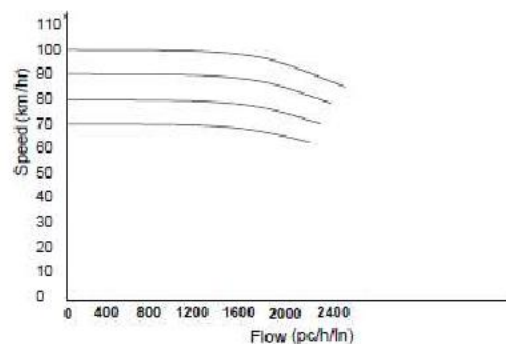
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1. Determination of free-flow speed
2. Determination of flow rate
3. Determination of level of service

Free-flow speed

Free-flow speed is the theoretical speed of traffic density, when density approaches zero. It is the speed at which drivers feel comfortable travelling under the physical, environmental and traffic conditions existing on an uncongested section of multilane highway. In practice, free-flow speed is determined by performing travel-time studies during periods of low-to-moderate flow conditions. The upper limit for low to moderate flow conditions is considered 1400 passenger cars per hour per lane (pc/h/ln) for the analysis. Speed-flow and density-flow relationships are shown in figure 2.11 and figure 2.12. These relationships hold for a typical uninterrupted-flow segment on a multilane highway under either base or no base conditions in which free-flow speed is known. Figure 2.11 indicates that the speed of traffic volume up to a flow rate of 1400 pc/h/ln. It also shows that the capacity of a multilane highway under base conditions is 2200 pc/h/ln for highways with a 90 km/h free-flow speed. At flow rates between 1400 and 2200 pc/h/ln, the speed on a multilane highway drops. Figure 2.12 shows that density varies continuously throughout the full range of flow rates. The capacity value of 2200 pc/h/ln is representative of the maximum 15-min flow rate that can be accommodated under base conditions for highways with 90 km/h free-flow speed. From various studies of the flow characteristics, base conditions for multilane highways are defined as follows:

1. Lane widths are 3.6 m.
2. Lateral clearance is 1.8 m.
3. A minimum of 3.6 m of total lateral clearance in the direction of travel. Clearances are measured from the edge of the outer travelled lanes (shoulders included) and lateral clearance of 1.8 m or greater are considered to be equal to 1.8 m.
4. No direct access points along the highway.
5. A divided highway.
6. Only passenger cars in the traffic stream.
7. A free-flow speed of 90 km/h or more.



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Figure 2.11 Speed-Flow relationship on multilane highway

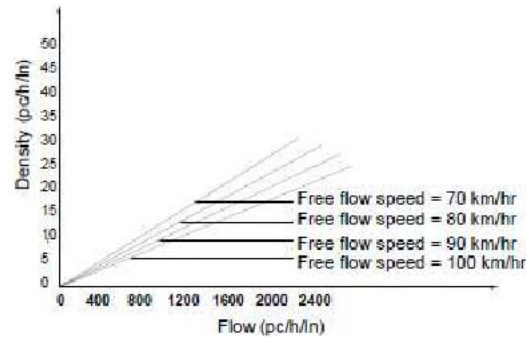


Figure 2.12 Density-flow relationships on multilane highways

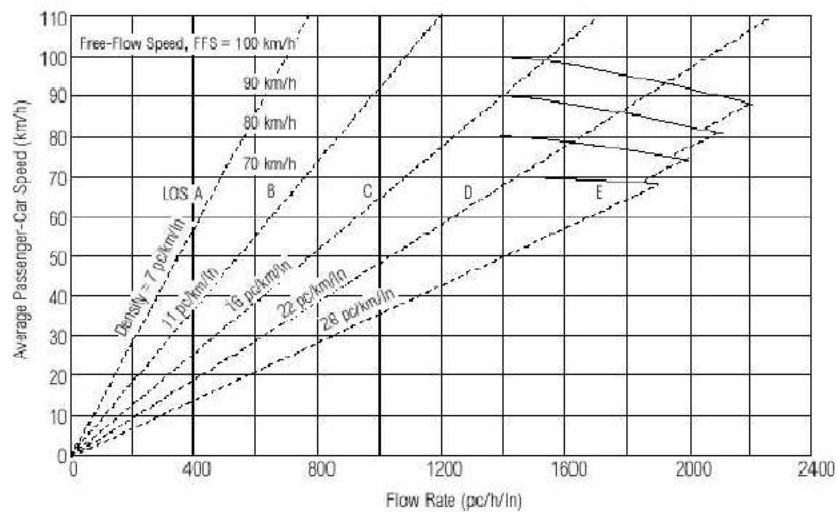


Figure 2.13 Speed-Flow curve with LOS Criteria for multilane highways

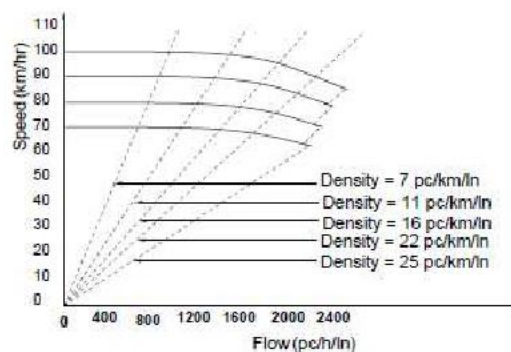


Figure 2.14 Flow chart showing step by step procedure to find density and LOS

The average of all passenger-car speeds measured in the field under low volume conditions can be used directly as the free-flow speed if such measurements were taken at flow rates at or below 1400 pc/h/ln. No adjustments are necessary as this speed reflects the net effect of all conditions at the site that influence speed, including lane width, lateral clearance, type of median, access points,

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posted speed limits, and horizontal and vertical alignment. Free-flow speed also can be estimated from 85th-percentile speed or posted speed limits, research suggests that free-flow speed under base conditions is 11 km/h higher than the speed limit for 65 km/h to 70 km/h speed limits and 8 km/h higher for 80 km/h to 90 km/h speed limits. Fig. 2.14 shows speed- flow curves with LOS criteria for multilane highways, here LOS is easily determined for any value of speed simply by plotting the point which is an intersection of flow and corresponding speed. Note that density is the primary determinant of LOS. LOS F is characterized by highly unstable and variable traffic flow. Prediction of accurate flow rate, density, and speed at LOS F is difficult.

Determination of free-flow speed

When field data are not available, the free-flow speed can be estimated indirectly as follows:

$$FFS = BFFS - f_{LW} - f_{LC} - f_m - f_A \quad \text{eq (1)}$$

where, FFS is the estimated FFS (km/h), $BFFS$ = base FFS (km/h), f_{LW} = adjustment for lane width, from Table 2.5 (km/h), f_{LC} = adjustment for lateral clearance, from Table 2.6 (km/h), f_m = adjustment for median type, from Table 2.7 (km/h), and f_A = adjustment for access points, from Table 2.8 (km/h). FFS on multilane highways under base conditions is approximately 11 km/h higher than the speed limit for 65 and 70 km/h speed limits, and it is 8 km/h higher for 80 and 90 km/h speed limits. $BFFS$ is approximately equal to 62.4 km/h (i.e decrease in 1.6 km/h) when the 85th percentile speed is 64 km/h, and it is 91.2 km/h (i.e decrease in 4.8 km/h) when the 85th percentile speed is 96 km/h and the in between speed values is found out by interpolation. According to Table 2.5, the adjustment in km/h increase as the lane width decreases from a base lane width of 3.6 m. No data exist for lane widths less than 3.0m.

The adjustment for lateral clearance (T_{LC}) is given as:

$$T_{LC} = L_{CL} + L_{CR}$$

where, T_{LC} = Total lateral clearance (m), L_{CL} = Lateral clearance (m), from the right edge of the travel lanes to roadside obstructions (if greater than 1.8 m, use 1.8 m), and L_{CR} = Lateral clearance (m), from the left edge of the travel lanes to obstructions in the roadway median (if the lateral clearance is greater than 1.8m, use 1.8 m). Once the total lateral clearance is computed, the adjustment factor is obtained from Table 2.6. For undivided highways, there is no adjustment for the right-side lateral clearance as this is already accounted for in the median type. Therefore, in order to use Table 2.7 for undivided highways, the lateral clearance on the left edge is always 1.8 m, as it for roadways with TWRTL's. The access-point density, which is use in Table 2.8, for a divided roadway is found by dividing the total number of access points (intersections and driveways) on the right side of the roadway in the direction of travel being studied by the length of the segment in kilometers. The adjustment factor for access-point density is given in Table 2.8. Thus the free flow speed can be computed using equation 1 and applying all the adjustment factors.

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Table 2.5 Adjustment for lane width (Source: HCM, 2000)

Lane Width (m)	Reduction in FFS(km/h)
3.6	0.0
3.5	1.0
3.4	2.1
3.3	3.1
3.2	5.6
3.1	8.1
3.0	10.6

Table 2.6 Adjustment for lateral clearance (Source: HCM, 2000)

Four-Lane Highways		Six-Lane Highways	
Total Lateral Clearance a (m)	Reduction in FFS (km/h)	Total Lateral Clearance a (m)	Reduction in FFS (km/h)
3.6	0.0	3.6	0.0
3.0	0.6	3.0	0.6
2.4	1.5	2.4	1.5
1.8	2.1	1.8	2.1
1.2	3.0	1.2	2.7
0.6	5.8	0.6	4.5
0.0	8.7	0.0	6.3

Table 2.7 Adjustment to free flow speed for median type (Source: HCM, 2000)

Median Type	Reduction in FFS (km/h)
Undivided highways	2.6
Divided highways	0.0

Table 2.8 Adjustment to free flow speed for Access-point density (Source: HCM, 2000)

Access Points/Kilometer	Reduction in FFS (km/h)
0	0.0
6	4.0
12	8.0
18	12.0
≥ 24	16.0

Determination of Flow Rate

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The next step in the determination of the LOS is the computation of the peak hour factor. The fifteen minute passenger-car equivalent flow rate (pc/h/ln), is determined by using following formula:

$$v_p = \frac{V}{(PHF \times N \times f_{HV} \times f_p)}$$

where, v_p is the 15-min passenger-car equivalent flow rate (pc/h/ln), V is the hourly volume (veh/h), PHF is the peak-hour factor, N is the number of lanes, f_{HV} is the heavy-vehicle adjustment factor, and f_p is the driver population factor. PHF represents the variation in traffic flow within an hour. Observations of traffic flow consistently indicate that the flow rates found in the peak 15-min period within an hour are not sustained throughout the entire hour. The PHF s for multilane highways have been observed to be in the range of 0.75 to 0.95. Lower values are typical of rural or off-peak conditions, whereas higher factors are typical of urban and suburban peak-hour conditions. Where local data are not available, 0.88 is a reasonable estimate of the PHF for rural multilane highways and 0.92 for suburban facilities. Besides that, the presence of heavy vehicles in the traffic stream decreases the FFS , because base conditions allow a traffic stream of passenger cars only. Therefore, traffic volumes must be adjusted to reflect an equivalent flow rate expressed in passenger cars per hour per lane (pc/h/ln). This is accomplished by applying the heavy-vehicle factor (f_{HV}). Once values for E_T and E_R have been determined, the adjustment factors for heavy vehicles are applied as follows:

$$f_{HV} = \frac{1}{(1 + P_T(E_T - 1) + P_R(E_R - 1))}$$

where, E_T and E_R are the equivalents for trucks and buses and for recreational vehicles (RVs), respectively, P_T and P_R are the proportion of trucks and buses, and RVs, respectively, in the traffic stream (expressed as a decimal fraction), f_{HV} is the adjustment factor for heavy vehicles. Adjustment for the presence of heavy vehicles in traffic stream applies for three types of vehicles: trucks, buses and recreational vehicles (RVs). Trucks cover a wide range of vehicles, from lightly loaded vans and panel trucks to the most heavily loaded coal, timber, and gravel haulers. An individual truck's operational characteristics vary based on the weight of its load and its engine performance. RVs also include a broad range: campers, self-propelled and towed; motor homes; and passenger cars or small trucks towing a variety of recreational equipment, such as boats, snowmobiles, and motorcycle trailers. There is no evidence to indicate any distinct differences between buses and trucks on multilane highways, and thus the total population is combined.

Table 2.9 Passenger-car equivalent on extended general highway segments (Source: HCM, 2000)

Factor	Type of Terrain		
	Level	Rolling	Mountainous
E_T (Trucks and Buses)	1.5	2.5	4.5
E_R (RVs)	1.2	2.0	4.0

Determination of Level of Service

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The level of service on a multilane highway can be determined directly from figure 2.14 or Table 2.10 based on the free-flow speed (FFS) and the service flow rate (v_p) in pc/h/ln.

Table 2.10 Level of Service criteria for a typical free flow speed of 100 kmph proposed in HCM 2000.

Free-Flow Speed	Criteria	(LOS) A	(LOS) B	(LOS) C	(LOS) D	(LOS) E
100 km/h	Max. density (pc/km/ln)	7	11	16	22	25
	Average speed (kmph)	100	100	98.4	91.5	88
	Max. volume capacity ratio	0.32	0.50	0.72	0.92	1.00
	Max. service flow rate (pc/h/ln)	700	1100	1575	2015	2200

The procedure is as follows:

1. Define a segment on the highway as appropriate. The following conditions help to define the segmenting of the highway,

- Change in median treatment
- Change in grade of 2% or more or a constant upgrade over 1220 m
- Change in the number of travel lanes
- The presence of a traffic signal
- A significant change in the density of access points
- Different speed limits
- The presence of bottleneck condition

In general, the minimum length of study section should be 760 m, and the limits should be no closer than 0.4 km from a signalized intersection.

2. On the basis of the measured or estimated free-flow speed on a highway segment, an appropriate speed-flow curve of the same as the typical curves is drawn.
3. Locate the point on the horizontal axis corresponding to the appropriate flow rate (v_p) in pc/hr/ln and draw a vertical line.
4. Read up the FFS curve identified in step 2 and determine the average travel speed at the point of intersection.
5. Determine the level of service on the basis of density region in which this point is located. Density of flow can be computed as

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$$D = \frac{v_p}{S}$$

where, D is the density (pc/km/ln), v_p is the flow rate (pc/h/ln), and S is the average passenger-car travel speed (km/h). The level of service can also be determined by comparing the computed density with the density ranges shown in table given by HCM. To use the procedures for a design, a forecast of future traffic volumes has to be made and the general geometric and traffic control conditions, such as speed limits, must be estimated. With these data and a threshold level of service, an estimate of the number of lanes required for each direction of travel can be determined.

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Numerical example 1

A segment of undivided four-lane highway on level terrain has field-measured FFS 74.0km/h, lane width 3.4-m, peak-hour volume 1,900-veh/h, 13 percent trucks and buses, 2 percent RVs, and 0.90 PHF. What is the peak-hour LOS, speed, and density for the level terrain portion of the highway?

Solution:

1. Data given:

- Level terrain, field measured FFS = 74 km/h,
- lane width is 3.4 m,
- peak-hour volume = 1900 veh/h,
- percent trucks and buses $P_T = 0.13$,
- percent RV's $P_R = 0.02$, and
- PHF=0.90.

2. Determination of flow rate (v_p):

LOS can be calculated by knowing flow rate and free flow speed.

Flow rate (v_p) is calculated from the equation

$$v_p = \frac{V}{(PHF \times N \times f_{HV} \times f_p)}$$

Since f_{HV} is unknown, it is calculated from the equation

$$f_{HV} = \frac{1}{(1 + P_T(E_T - 1) + P_R(E_R - 1))}$$

where, E_T and E_R are passenger-car equivalents for trucks and buses and for recreational vehicles (RVs) respectively P_T and P_R are proportion of trucks and buses, and RVs, respectively, in the traffic stream (expressed as a decimal fraction)

$$f_{HV} = \frac{1}{1 + 0.13(1.5 - 1) + 0.02(1.2 - 1)}$$

Assume no RVs, since none is indicated.

$$V_p = \frac{1900}{(0.90 * 2 * 0.935 * 1)}$$

$$= 1129 \text{ pc/hr/ln}$$

3. Determination of free flow speed(S):

In this example the free flow speed (FFS) measured at the field is given and hence no need to compute free flow speed by indirect method. Therefore, $\text{FFS} = S = 74.0 \text{ km/h}$.

4. Determination of density(D):

The density of flow is computed from the equation

$$D = \frac{V_p}{S} = 15.3$$

5. Determination of LOS:

LOS determined from the speed-flow diagram. $\text{LOS} = \text{C}$.

Numerical example 2

A segment of an east-west five-lane highway with two travel lanes in each direction separated by a two-way left-turn lane (TWLTL) on a level terrain has 83.0-km/h 85th-percentile speed, 3.6-m lane width, 1,500-veh/h peak-hour volume, 6 % trucks and buses, 8 access points/km (WB), 6 access points/km (EB), 0.90 PHF, 3.6-m and greater lateral clearance for westbound and eastbound. What is the LOS of the highway on level terrain during the peak hour?

Solution:

1. Data given:

Level terrain,

85th-percentile speed is 83.0 km/h,

lane width is 3.6 m,

peak-hour volume, $V=1500 \text{ veh/h}$

Proportion of trucks and buses $P_T= 0.06$,

8 access points/km in WB,

6 access points/km in EB,

$PHF = 0.90$, and

Lateral clearance for westbound and eastbound is more than 3.6 m.

2. Determination of flow rate (v_p):

LOS can be calculated by knowing flow rate and free flow speed. Flow rate (v_p) is calculated from the equation

$$v_p = \frac{V}{(PHF \times N \times f_{HV} \times f_p)}$$

where, v_p = 15-min passenger-car equivalent flow rate (pc/h/ln), V = hourly volume (veh/h), PHF = peak-hour factor, N = number of lanes, f_{HV} = heavy-vehicle adjustment factor, and f_p = driver population factor Since f_{HV} is unknown it is calculated from the equation

$$f_{HV} = \frac{1}{(1 + P_T(E_T - 1) + P_R(E_R - 1))}$$

where, E_T and E_R = passenger-car equivalents for trucks and buses and for recreational vehicles (RVs), respectively P_T and P_R = proportion of trucks and buses, and RVs, respectively, in the traffic stream (expressed as a decimal fraction) Assume no RVs, since none is indicated.

$$f_{HV} = \frac{1}{1 + 0.06(1.5 - 1) + 0}$$

$$= 0.970$$

$$V_p = \frac{1500}{(0.90 * 2 * 0.970 * 1)}$$

3. Determination of free flow speed(S):

BFFS is approximately equal to 62.4 km/h when the 85th percentile speed is 64 km/h, and it is 91.2 km/h when the 85th percentile speed is 96 km/h and the in between speed values is found out by interpolation. Hence,

BFFS = 80 km/h.

Now, compute east bound and west bound free-flow speeds

$$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_m$$

$$= 80 - 0 - 0 - 4 - 0$$

$$= 76 \text{ kmph (WB)}$$

$$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_m$$

$$= 80 - 0 - 0 - 5.3 - 0$$

$$= 74.7 \text{ kmph (EB)}$$

4. Determination of LOS: LOS determined from the speed-flow diagram. LOS = C (for EB) LOS = C (for WB)

Numerical example 3

A 10 km long 4 lane undivided multilane highway in a suburban area has a segment 1 km long with a 3% upgrade and a segment 1 km long with a 3% downgrade. The section has a volume of 1900 vehicles/hr in each direction with 13% trucks and buses and 2% recreational vehicles. The 85th percentile speed of passenger car is 80 km/hr on upgrade and 86km/hr on downgrade. There are total of 12 access points on both sides of the roadway. The lane width is 3.6 m, PHF is 0.90 and having a 3m lateral clearance. Determine the LOS of the highway section (upgrade and downgrade) during the peak hour? From HCM, For a 3% upgrade and 1 km length (ET=1.5 , ER=3) For a 3% downgrade and 1 km length(ET=1.5 , ER=1.2)

Solution:

1. Data given:

3% upgrade and 3% downgrade

No. of lanes = 4, N = 4,

80.0 km/h 85th percentile speed for upgrade,

86 km/h 85th percentile speed for downgrade,

3.6m lane width,

1,900-veh/h peak-hour volume, (V = 1900 veh/h)

13 % trucks and buses, (P_T=0.13)

2 % Recreational vehicles, (P_R=0.02)

12 access points/km,

$$PHF = 0.90$$

$$\text{lateral clearance} = 3 \text{ m}$$

2. Determination of flow rate (v_p):

LOS can be calculated by knowing flow rate and free flow speed.

For upgrade: Flow rate is calculated from the equation

$$v_p = \frac{V}{(PHF \times N \times f_{HV} \times f_p)}$$

where, v_p = 15-min passenger-car equivalent flow rate (pc/h/ln), V = hourly volume (veh/h), PHF = peak-hour factor, N = number of lanes, f_{HV} = heavy-vehicle adjustment factor, and f_p = driver population factor Since f_{HV} is unknown it is calculated from the equation

$$f_{HV} = \frac{1}{(1 + P_T(E_T - 1) + P_R(E_R - 1))}$$

where, E_T and E_R = passenger-car equivalents for trucks and buses and for recreational vehicles (RVs), respectively P_T and P_R = proportion of trucks and buses, and RVs, respectively, in the traffic stream (expressed as a decimal fraction) Assume no RVs, since none is indicated.

$$f_{HV} = \frac{1}{1 + 0.13(1.5 - 1) + 0.02(3 - 1)} = 0.905$$

$$V_p = \frac{1900}{0.90 \times 2 \times 0.905 \times 1} = 1166 \text{ pc/hr/ln}$$

For downgrade:

$$f_{HV} = \frac{1}{1 + 0.13(1.5 - 1) + 0.02(1.2 - 1)} = 0.935$$

$$V_p = \frac{1900}{0.90 \times 2 \times 0.935 \times 1} = 1128 \text{ pc/hr/ln}$$

3. Determination of free flow speed(S):

For upgrade: BFFS is approximately equal to 62.4 km/h when the 85th percentile speed is 64 km/h, and it is 91.2 km/h when the 85th percentile speed is 96 km/h and the in between speed values is found out by interpolation. Hence for 86 km/hr 85th percentile speed from interpolation we get, BFFS = 77.0 km/h Now, Compute east bound and west bound free-flow speeds

$$FFS = BFFS - f_{LW} - f_{LC} - f_m - f_A$$

$$= 77 - 0 - 0.6 - 8.0 - 2.6$$

$$= 65.8 \text{ kmph}$$

For downgrade:

BFFS is approximately equal to 62.4 km/h when the 85th percentile speed is 64 km/h, and it is 91.2 km/h when the 85th percentile speed is 96 km/h and the in between speed values is found out by interpolation. Hence for 86 km/hr 85th percentile speed from interpolation we get, BFFS= 82.0 km/h Now, Compute the free-flow speed

$$FFS = BFFS - f_{LW} - f_{LC} - f_m - f_A$$

$$= 82 - 0 - 0.6 - 8.0 - 2.6$$

$$= 71 \text{ kmph}$$

4. Determination of LOS

LOS determined from the speed-flow diagram. LOS = D (for upgrade) LOS = D (for downgrade)

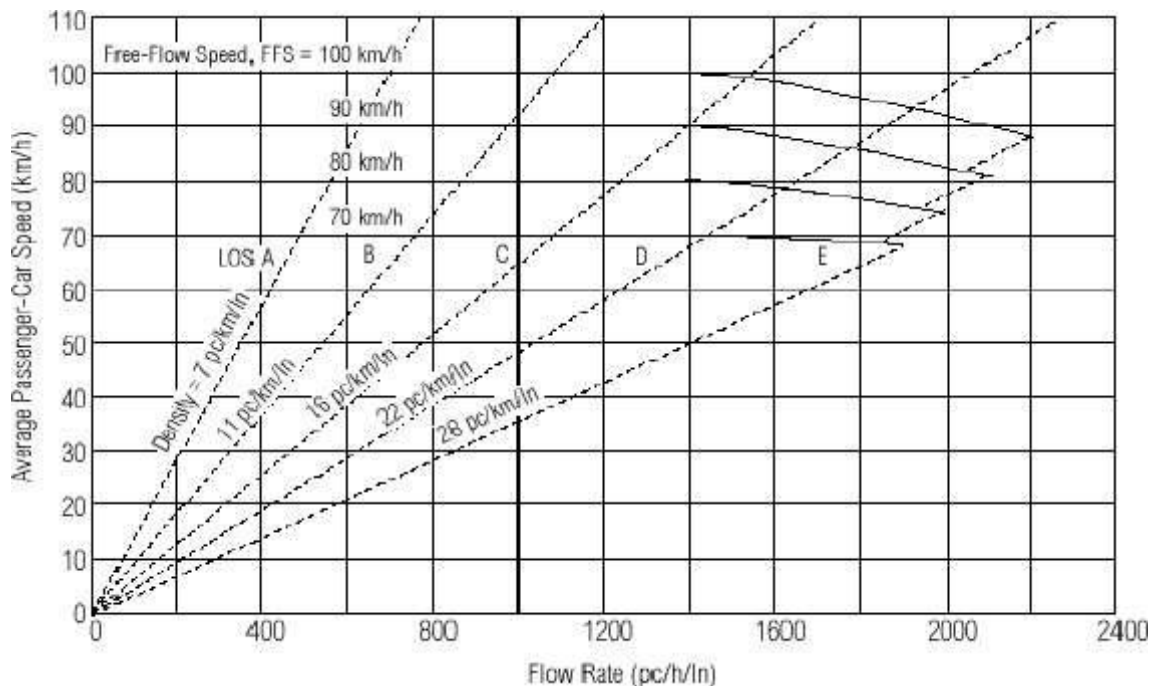


Figure: Speed-Flow curve with LOS Criteria for multilane highways

PARKING STUDIES

3.1 General

Parking is one of the major problems that is created by the increasing road traffic. It is an impact of transport development. The availability of less space in urban areas has increased the demand for parking space especially in areas like Central business district (CBD). This affects the mode choice also. This has a great economic impact.

3.1.1 Ill effects of parking

Parking has some ill-effects like congestion, accidents, pollution, obstruction to fire-fighting operations etc.

1. **Congestion:** Parking takes considerable street space leading to the lowering of the road capacity. Hence, speed will be reduced, journey time and delay will also subsequently increase.
2. **Accidents:** Careless manoeuvring of parking and unparking leads to accidents which are referred to as parking accidents. Common type of parking accidents occur while driving out a car from the parking area, careless opening of the doors of parked cars, and while bringing in the vehicle to the parking lot for parking.
3. **Environmental pollution:** They also cause pollution to the environment because stopping and starting of vehicles while parking and unparking results in noise and fumes. They also affect the aesthetic beauty of the buildings because cars parked at every available space creates a feeling that building rises from a plinth of cars.
4. **Obstruction to fire fighting operations:** Parked vehicles may obstruct the movement of fire fighting vehicles. Sometimes they block access to hydrants and access to buildings.

3.1.2 Types of parking facilities

3.1.2.1 On-street parking

On-street parking means the vehicles are parked on the sides of the street itself. This will be usually controlled by government agencies itself. Common types of on-street parking are as listed below. This classification is based on the angle in which the vehicles are parked with respect to the road alignment. As per IRC the standard dimensions of a car is taken as 5×2.5 meters and that for a truck is 3.75×7.5 meters.

1. **Parallel parking:** The vehicles are parked along the length of the road. Here there is no backward movement involved while parking or unparking the vehicle. Hence, it is the safest parking from the accident perspective. However, it consumes the maximum kerb length and therefore only a minimum number of vehicles can be parked for a given kerb length. This method of parking produces least obstruction to the on-going traffic on the road since least road width is used. Parallel parking of cars is shown in figure 3.1.

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Figure 3.1: Illustration of parallel parking

The length available to park N number of vehicles, $L = N * 5.9$

2. **30° parking:** In thirty degree parking, the vehicles are parked at 30° with respect to the road alignment. In this case, more vehicles can be parked compared to parallel parking. Also there is better manoeuvrability. Delay caused to the traffic is also minimum in this type of parking. An example is shown in figure 3.2. From the figure,

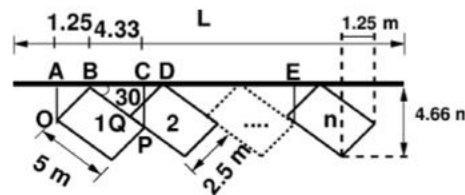


Figure 3.2: Illustration of 30° parking

$$AB = OB \sin 30 = 1.25$$

$$BC = OP \cos 30 = 4.33$$

$$BD = DQ \cos 60 = 5$$

$$CD = BD - BC = 5 - 4.33 = 0.67$$

$$AB + BC = 1.25 + 4.33 = 5.58$$

$$\text{For } N \text{ vehicles, } L = AC + (N - 1)E$$

$$= 5.58 + (N - 1)5$$

$$\text{Therefor, } L = 5N + 0.58$$

3. **45° parking:** As the angle of parking increases, more number of vehicles can be parked. Hence compared to parallel parking and thirty degree parking, more number of vehicles can be accommodated in this type of parking. From figure 3.3, length of parking space available for parking N number of vehicles in a given kerb is $L = 3.54N + 1.77$

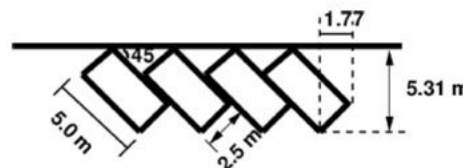


Figure 3.3: Illustration of 45° parking

4. **60° parking:** The vehicles are parked at 60° to the direction of road. More number of vehicles can be accommodated in this parking type. From the figure 3.4, length available for parking N vehicles, $L = 2.89N + 2.16$

UNIT - III PARKING STUDIES AND TRAFFIC ACCIDENT STUDIES

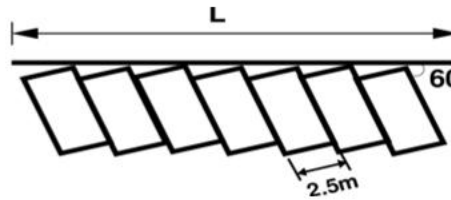


Figure 3.4: Illustration of 60° parking

5. **Right angle parking:** In right angle parking or 90° parking, the vehicles are parked perpendicular to the direction of the road. Although it consumes maximum width kerb length required is very little. In this type of parking, the vehicles need complex manoeuvring and this may cause severe accidents. This arrangement causes obstruction to the road traffic particularly if the road width is less. However, it can accommodate maximum number of vehicles for a given kerb length. An example is shown in figure 3.5. Length available for parking N number of vehicles is $L = 2.5N$

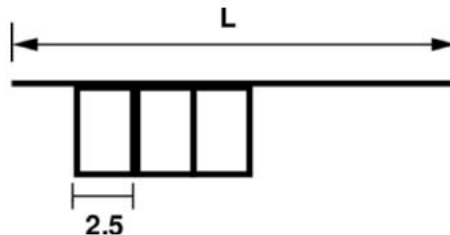


Figure 3.5: Illustration of 90° parking

3.1.2.2 Off-street parking

In many urban centers, some areas are exclusively allotted for parking which will be at some distance away from the main stream of traffic. Such a parking is referred to as off-street parking. They may be operated by either public agencies or private firms. A typical layout of an off-street parking is shown in figure 3.6.

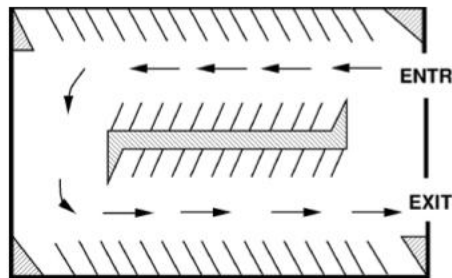


Figure 3.6: Illustration of off-street parking

It includes (a). Surface car parking, (b). Multi-storey car parking, (c). Roof parking, (d). Mechanical car parking, (e). Underground car parking.

- Surface car parks: This type of parking is used at super market, complex & office.
- Multi-storey car parks: This type of parking design for 400 – 500 car parking. In this type of parking, parking design for max 4 – 5 floor. This parking is use for large area.

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- Roof parks: Because of less space in many cities parking facility provided on roof of the building. For transportation of vehicle arrangement of ramp or mechanical lift is there.
- Mechanical car parks: In this method with the help of lift cars are lifted from one floor to another floor. Cars are parked in stall with the help of mechanical trolley. Maintenance cost is more.
- Underground car parks: Basement of building.
 - Costly: foundation
 - Ventilation
 - Lighting

3.2 Parking studies and analysis

3.2.1 Parking requirements

There are some minimum parking requirements for different types of building. For residential plot area less than 300sq.m require only community parking space. For residential plot area from 500 to 1000sq.m, minimum one-fourth of the open area should be reserved for parking. Offices may require at least one space for every 70sq.m as parking area. One parking space is enough for 10 seats in a restaurant where as theatres and cinema halls need to keep only 1 parking space for 20 seats. Thus, the parking requirements are different for different land use zones.

3.2.2 Parking statistics

Parking surveys are intended to provide all these information. Since the duration of parking varies with different vehicles, several statistics are used to access the parking need. The following parking statistics are normally important.

1. **Parking accumulation:** It is defined as the number of vehicles parked at a given instant of time. Normally this is expressed by accumulation curve. Accumulation curve is the graph obtained by plotting the number of bays occupied with respect to time.
2. **Parking volume:** Parking volume is the total number of vehicles parked at a given duration of time. This does not account for repetition of vehicles. The actual volume of vehicles entered in the area is recorded.
3. **Parking load:** Parking load gives the area under the accumulation curve. It can also be obtained by simply multiplying the number of vehicles occupying the parking area at each time interval with the time interval. It is expressed as vehicle hours.
4. **Average parking duration:** It is the ratio of total vehicle hours to the number of vehicles parked.

$$\text{parking duration} = \frac{\text{parking load}}{\text{parking volume}} \quad \text{eq(1)}$$

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5. **Parking turnover:** It is the ratio of number of vehicles parked in a duration to the number of parking bays available. This can be expressed as number of vehicles per bay per time duration.

$$\text{parking turnover} = \frac{\text{parking volume}}{\text{no.of bays available}} \quad \text{eq(2)}$$

6. **Parking index:** Parking index is also called occupancy or efficiency. It is defined as the ratio of number of bays occupied in a time duration to the total space available. It gives an aggregate measure of how effectively the parking space is utilized. Parking index can be found out as follows

$$\text{parking index} = \frac{\text{parking load}}{\text{parking capacity}} \times 100 \quad \text{eq (3)}$$

3.3 Parking surveys

Parking surveys are conducted to collect the above said parking statistics. The most common parking surveys conducted are in-out survey, fixed period sampling and license plate method of survey.

3.3.1 In-out survey

In this survey, the occupancy count in the selected parking lot is taken at the beginning. Then the number of vehicles that enter the parking lot for a particular time interval is counted. The number of vehicles that leave the parking lot is also taken. The final occupancy in the parking lot is also taken. Here the labour required is very less. Only one person may be enough. But we won't get any data regarding the time duration for which a particular vehicle used that parking lot. Parking duration and turnover is not obtained. Hence we cannot estimate the parking fare from this survey. For quick survey purposes, a fixed period sampling can also be done. This is almost similar to in-out survey. All vehicles are counted at the beginning of the survey. Then after a fixed time interval that may vary between 15 minutes to 1 hour, the count is again taken. Here there are chances of missing the number of vehicles that were parked for a short duration.

3.3.2 License plate method of survey

This results in the most accurate and realistic data. In this case of survey, every parking stall is monitored at a continuous interval of 15 minutes or so and the license plate number is noted down. This will give the data regarding the duration for which a particular vehicle was using the parking bay. This will help in calculating the fare because fare is estimated based on the duration for which the vehicle was parked. If the time interval is shorter, then there are less chances of missing short-term parkers. But this method is very labour intensive.

ACCIDENT STUDIES

3.4 General

The problem of accident is a very acute in highway transportation due to complex flow pattern of vehicular traffic, presence of mixed traffic along with pedestrians. Traffic accident leads to loss of life and property. Thus the traffic engineers have to undertake a big responsibility of

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providing safe traffic movements to the road users and ensure their safety. Road accidents cannot be totally prevented but by suitable traffic engineering and management the accident rate can be reduced to a certain extent. For this reason systematic study of traffic accidents are required to be carried out. Proper investigation of the cause of accident will help to propose preventive measures in terms of design and control.

3.4.1 Objectives of accident studies

Some objectives of accident studies are listed below:

1. To study the causes of accidents and suggest corrective measures at potential location
2. To evaluate existing design
3. To compute the financial losses incurred
4. To support the proposed design and provide economic justification to the improvement suggested by the traffic engineer
5. To carry out before and after studies and to demonstrate the improvement in the problem.

3.4.2 Causes of road accidents

The various causes of road accidents are:

1. **Road Users** - Excessive speed and rash driving, violation of traffic rules, failure to perceive traffic situation or sign or signal in adequate time, carelessness, fatigue, alcohol, sleep etc.
2. **Vehicle** - Defects such as failure of brakes, steering system, tyre burst, lighting system.
3. **Road Condition** - Skidding road surface, pot holes, ruts.
4. **Road design** - Defective geometric design like inadequate sight distance, inadequate width of shoulders, improper curve design, improper traffic control devices and improper lighting,.
5. **Environmental factors** - Unfavourable weather conditions like mist, snow, smoke and heavy rainfall which restrict normal visibility and makes driving unsafe.
6. **Other causes** - Improper location of advertisement boards, gate of level crossing not closed when required etc.

3.5 Accident Analysis

3.5.1 Accident data collection

The accident data collection is the first step in the accident study. The data collection of the accidents is primarily done by the police. Motorist accident reports are secondary data which are filed by motorists themselves. The data to be collected should comprise all of these parameters:

1. **General** - Date, time, person involved in accident, classification of accident like fatal, serious, minor
2. **Location** - Description and detail of location of accident
3. **Details of vehicle involved** - Registration number, description of vehicle, loading detail, vehicular defects
4. **Nature of accident** - Details of collision, damages, injury and casualty

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5. **Road and traffic condition** - Details of road geometry, surface characteristics, type of traffic, traffic density etc.
6. **Primary causes of accident** - Details of various possible cases which are the main causes of accident.
7. **Accident cost** - Financial losses incurred due to property damage, personal injury and casualty

These data collected need proper storing and retrieving for the following purpose. The purposes are as follows:

1. Identification of location of points at which unusually high number of accident occur.
2. Detailed functional evaluation of critical accident location to identify the causes of accidents.
3. Development of procedure that allows identification of hazards before large number of accidents occurs.
4. Development of different statistical measures of various accident related factors to give insight into general trends, common casual factors, driver profiles, etc.

3.5.2 Accident investigation

The accident data collection involves extensive investigation which involves the following procedure:

1. **Reporting:** It involves basic data collection in form of two methods:
 - a. **Motorist accident report** - It is filed by the involved motorist involved in all accidents fatal or injurious.
 - b. **Police accident report** - It is filed by the attendant police officer for all accidents at which an officer is present. This generally includes fatal accidents or mostly accidents involving serious injury required emergency or hospital treatment or which have incurred heavy property damage.
2. **At Scene-Investigation:** It involves obtaining information at scene such as measurement of skid marks, examination of damage of vehicles, photograph of final position of vehicles, examination of condition and functioning of traffic control devices and other road equipment's.
3. **Technical Preparation:** This data collection step is needed for organization and interpretation of the study made. In this step measurement of grades, sight distance, preparing drawing of after accident situation, determination of critical and design speed for curves is done.
4. **Professional Reconstruction:** In this step effort is made to determine from whatever data is available how the accident occurs from the available data. It is professionally referred as determining behavioral or mediate causes of accident.
5. **Cause Analysis:** It is the effort made to determine why the accident occurred from the data available and the analysis of accident reconstruction studies.

3.5.3 Accident data analysis

The purpose is to find the possible causes of accident related to driver, vehicle, and roadway. Accident analyses are made to develop information such as:

1. Driver and Pedestrian - Accident occurrence by age groups and relationships of accidents to physical capacities and to psychological test results.

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2. Vehicle - Accident occurrence related to characteristic of vehicle, severity, location and extent of damage related to vehicles.
3. Roadway conditions - Relationships of accident occurrence and severity to characteristics of the roadway and roadway condition and relative values of changes related to roadways.

It is important to compute accident rate which reflect accident involvement by type of highway. These rates provide a means of comparing the relative safety of different highway and street system and traffic controls. Another is accident involvement by the type of drivers and vehicles associated with accidents.

1. Accident Rate per Kilometer:

On this basis the total accident hazard is expressed as the number of accidents of all types per km of each highway and street classification.

$$R = \frac{A}{L} \quad (1)$$

where, R = total accident rate per km for one year, A = total number of accident occurring in one year, L = length of control section in kms

2. Accident involvement Rate:

It is expressed as numbers of drivers of vehicles with certain characteristics who were involved in accidents per 100 million vehicle-kms of travel.

$$R = \frac{N \times 100000000}{V} \quad (2)$$

where, R = accident involvement per 100 million vehicle-kms of travel, N = total number of drivers of vehicles involved in accidents during the period of investigation and V = vehicle-kms of travel on road section during the period of investigation

3. Death rate based on population:

The traffic hazard to life in a community is expressed as the number of traffic fatalities per 100,000 populations. This rate reflects the accident exposure for entire area.

$$R = \frac{B \times 100000}{P} \quad (3)$$

where, R = death rate per 100,000 population, B = total number of traffic death in one year and P = population of area

4. Death rate based on registration:

The traffic hazard to life in a community can also be expressed as the number of traffic fatalities per 10,000 vehicles registered. This rate reflects the accident exposure for entire area and is similar to death rate based on population.

$$R = \frac{B \times 10000}{M} \quad (4)$$

where, R = death rate per 10,000 vehicles registered, B = total number of traffic death in one year and M = number of motor vehicles registered in the area

5. Accident Rate based on vehicle-kms of travel:

The accident hazard is expressed as the number of accidents per 100 million vehicle km of travel. The true exposure to accident is nearly approximated by the miles of travel of the motor vehicle than the population or registration.

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$$R = \frac{C \times 100000000}{V} \quad (5)$$

where, R = accident rate per 100 million vehicle kms of travel, C = number of total accidents in one year and V = vehicle kms of travel in one year

3.6 Safety measures

The ultimate goal is to develop certain improvement measures to mitigate the circumstances leading to the accidents. The measures to decrease the accident rates are generally divided into three groups engineering, enforcement and education. Some safety measures are described below:

3.6.1 Safety measures related to engineering

The various measures of engineering that may be useful to prevent accidents are enumerated below

1. Visual guidance to driver

There is consecutive change of picture in drivers mind while he is in motion. The number of factors that the driver can distinguish and clearly fix in his mind is limited. So using the laws of visual perception certain measures have been suggested:

1. Contrast in visibility of the road should be achieved by provision of elements that differ from its surrounding by colors, pattern such as shoulder strips, shoulder covered with grass, edge markings.
2. Providing road side vegetation is an effective means.
3. The visibility of crown of trees from a distant location is also very useful in visual guiding.
4. The provision of guard rails of different contrasting colors also takes drivers attention and prevent from monotonous driving.

Figure 3.8 and 3.9 is a visual guidance measure. Planting trees along side of roadway which has a turning angle attracts attention of the driver and signals that a turn is present ahead.



Figure 3.8 Bifurcation of the highway

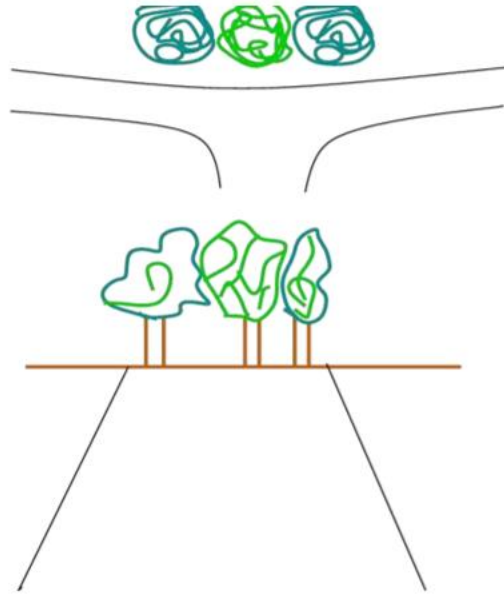


Figure 3.9 Road seemed to be stopped by a dense forest

2. Road reconstruction

The number of vehicles on the road increases from year to year, which introduces complications into organization of traffic, sharply reduces the operation and transportation characteristic of roads and lead to the growth of accident rate. This leads to the need of reconstructing road. The places of accidents need to be properly marked so that the reconstruction can be planned accordingly.

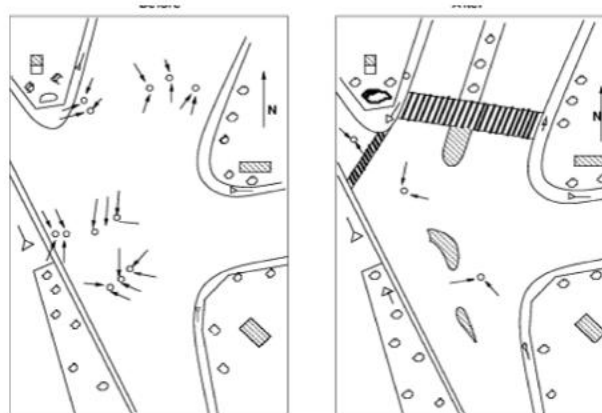


Figure 3.10 Diagram of accidents before and after reconstruction

The Figure 3.10 shows that there were too many conflict points before which reduced to a few number after construction of islands at proper places. Reconstruction process may also include construction of a new road next to the existing road, renewal of pavement without changing the horizontal alignment or profile of the road, reconstruction a particular section of road.

3. Channelization

The Channelization of traffic at intersection separates the traffic stream travelling in different direction, providing them a separate lane that corresponds to their convenient path and spreading as far as possible the points of conflict between crossing traffic streams. The traffic

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lanes are separated by marking relevant lines or by constructing slightly elevated islands as shown in Figure 3.12.

The principles of proper channelized intersection are:-

1. The layout of intersection should be visibly clear, simple and understandable by driver.
2. Should ensure superiority to the vehicles using road of higher class.
3. Layout of intersection makes it necessary for a driver running through it to choose at each moment of time one of not more than two possible direction of travel. This is achieved by visual guidance, islands and markings.
4. The island provided should separate high speed, through and turning traffic flows.
5. The width of traffic lane should ensure unhampered turning to the big vehicles.
6. Pedestrian crossing should be provided

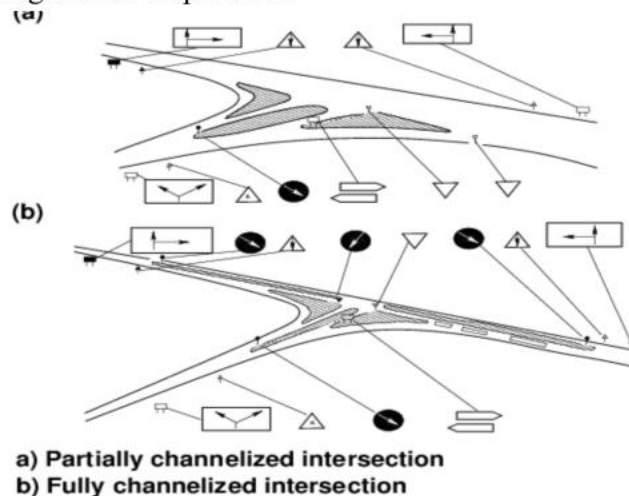


Figure 3.12 Channelized Intersection ensuring safety

4. Road signs

Road signs are integral part of safety as they ensure safety of the driver himself (warning signs) and safety of the other vehicles and pedestrians on road (regulatory signs). Driver should be able to read the sign from a distance so that he has enough time to understand and respond. It is essential that they are installed and have correct shape, colour, size and location.

Other methods

Various other methods of traffic accident mitigation are described below:

1. **Street lighting:** Street lightning of appropriate standard contributes to safety in urban area during night time due to poor visibility.
2. **Improvement in skid resistance:** If road is very smooth then skidding of the vehicles may occur or if the pavement is wet then wet weather crashes occur. Thus it is important to improve the skid resistance of the road. Various ways of increasing the skid resistance of road are by constructing high-friction overlay or cutting of grooves into the pavement.
3. **Road markings:** Road markings ensure proper guidance and control to the traffic on a highway. They serve as supplementary function of road sign. They serve as psychological barrier and delineation of traffic path and its lateral clearance from traffic

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hazards for the safe movement of traffic. Thus their purpose is to provide smooth and safe traffic flow.

4. **Guide posts with or without reflector:** They are provided at the edge of the roadway to prevent the vehicles from being off tracked from the roadway. Their provision is very essential in hilly road to prevent the vehicle from sliding from top. Guide posts with reflector guide the movement of vehicle during night.
5. **Guard rail:** Guard rail have similar function as of guide post. On high embankments, hilly roads, road running parallel to the bank of river, shores of lake, near rock protrusion, trees, bridge, abutments a collision with which is a great hazard for a vehicle. It is required to retain the vehicle on the roadway which has accidentally left the road because of fault or improper operation on the part of the driver. Driver who has lost control create a major problem which can be curbed by this measure.
6. **Driver reviver stop:** Driver reviver stop are generally in use in countries like U.S.A where driver can stop and refresh himself with food, recreation and rest. They play a very important part in traffic safety as they relieve the driver from the mental tension of constant driving. These stops are required to be provided after every 2 hour travel time.
7. **Constructing flyovers and bypass:** In areas where local traffic is high bypasses are required to separate through traffic from local traffic to decrease the accident rate. To minimize conflicts at major intersections flyovers are required for better safety and less accident rate
8. **Regular accident studies:** Based on the previous records of accidents the preventive measures are taken and after that the data related to accidents are again collected to check the efficiency of the measures and for future implementation of further preventive measures.

3.6.2 Safety measures related to enforcement

The various measures of enforcement that may be useful to prevent accidents at spots prone to accidents are enumerated below. These rules are revised from time to time to make them more comprehensive.

1. Speed control

Checks on spot speed of all vehicles should be done at different locations and timings and legal actions on those who violate the speed limit should be taken

2. Training and supervision

The transport authorities should be strict while issuing licence to drivers of public service vehicles and taxis. Driving licence of the driver may be renewed after specified period, only after conducting some tests to check whether the driver is fit

3. Medical check

The drivers should be tested for vision and reaction time at prescribed intervals of time

3.6.3 Safety measures related to education

The various measures of education that may be useful to prevent accidents are enumerated below.

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1. Education of road users

The passengers and pedestrians should be taught the rules of the road, correct manner of crossing etc. by introducing necessary instruction in the schools for the children and by the help of posters exhibiting the serious results due to carelessness of road users.

2. Safety drive

Imposing traffic safety week when the road users are properly directed by the help of traffic police as a means of training the public. Training courses and workshops should be organized for drivers in different parts of the country.

3.6.4 Safety audit

It is the procedure of assessment of the safety measures employed for the road. It has the advantages like proper planning and decision from beforehand ensures minimization of future accidents, the long term cost associated with planning is also reduced and enables all kinds of users to perceive clearly how to use it safely. Safety audit takes place in five stages as suggested by Wrisberg and Nilsson, 1996. Five Stages of Safety Audit are:

1. **Feasibility Stage** - The starting point for the design is determined such as number and type of intersection, relationship of the new scheme to the existing road, the relevant design standards.
2. **Draft Stage** - In this stage horizontal and vertical alignment, junction layout are determined. After the completion of this stage decision about land acquisition is taken.
3. **Detailed design stage** - Signing, marking, lighting, other roadside equipment and landscaping are determined.
4. **Pre-opening stage** - Before opening a new or modified road should be driven, walked or cycled. It should be done at different condition like bad weather, darkness.
5. **Monitoring of the road in use** - Assessment is done at the final stage after the road has been in operation for few months to determine whether the utilization is obtained as intended and whether any adjustment to the design are required in the light of the actual behavior of road users.

References

1. https://www.civil.iitb.ac.in/tvm/nptel/581_Parking/web/web.html
2. https://www.civil.iitb.ac.in/tvm/nptel/582_Accident/web/web.html

Unit-IV Intersection Design

4.0 Intersection

The term junction or intersection describes all roadway situations where two or more roads meet or cross one another at the same or different elevations.

The design and layout of road junctions demand careful consideration from the view point of traffic management because it is very essential to have safety of vehicular traffic and pedestrians at such places. The accidents generally occur at road junctions and hence, proper precautions should be taken in their design and layout. They should be properly signalled and it is essential to have proper segregation and control of the traffic.

4.1 Uncontrolled intersection: Are the most common type of intersection usually occurs where the intersecting roads are relatively equal importance and found in areas where there is not much traffic shown in Figure 4.1. Uncontrolled intersections are the traffic junctions where there is no explicit traffic control measures are adopted.

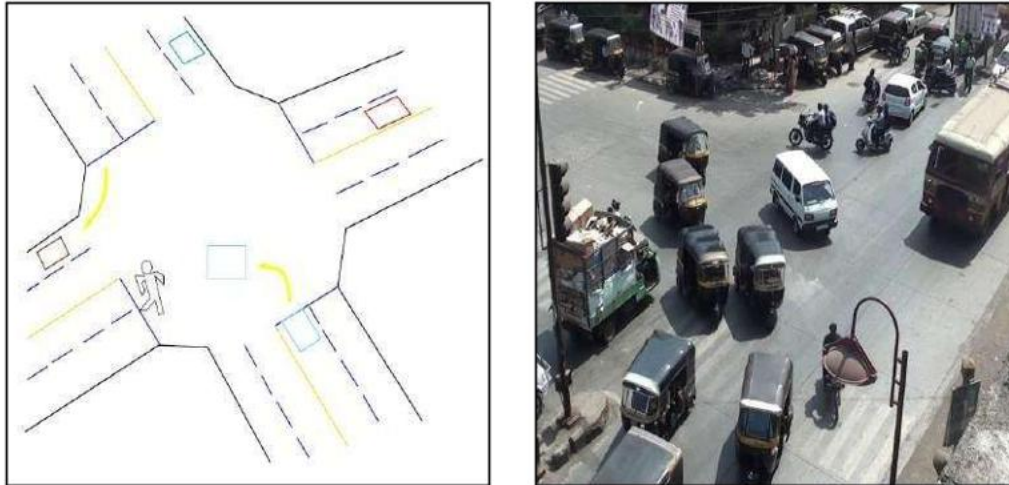


Figure 4.1: Examples showing uncontrolled intersection

4.2 Definitions and Important Terms

1. Channelization - It is the separation or regulation of conflicting traffic movements into definite paths of travel by traffic islands or pavement marking to facilitate the safe and orderly movements of both vehicles and pedestrians.
2. Conflict - It is defined as the demand for the same highway space by two or more users of the highway. Conflicts are classified into mainly three types:
 - (a) Crossing conflicts
 - (b) Diverging conflicts
 - (c) Merging conflicts
3. Angle of Intersection - The angle of intersection is that formed by the centerlines of the intersecting streets. Where the angle of intersection departs significantly (more than approximately 20°) from right angles, the intersection is referred to as a skewed intersection. Figure 4.2 shows the angle made between the center lines of the major and minor legs.
4. Refuge Areas - The area which is used to give refuge to the pedestrians crossing a street (the open area between two medians) is known as a refuge area.

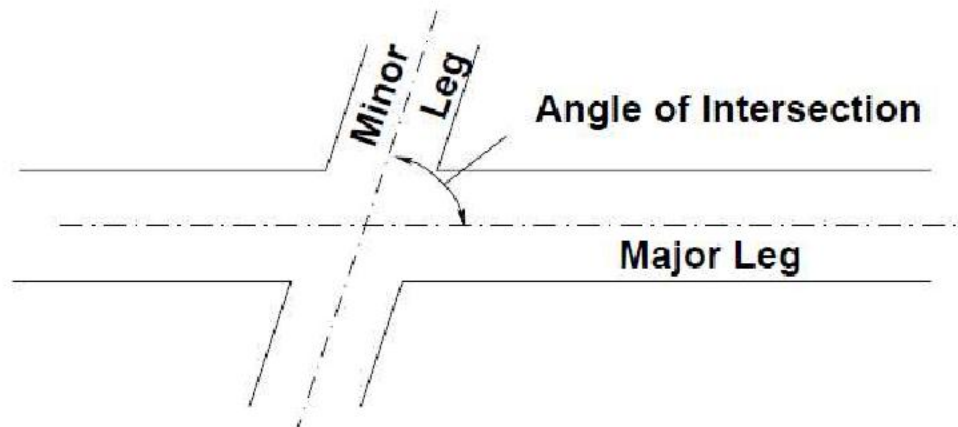


Figure 4.2: Angle of Intersection

4.3 Conflicts at an intersection

Conflicts at an intersection are different for different types of intersection. Consider a typical four-legged intersection as shown in figure. The number of conflicts for competing through movements are 4, while competing right turn and through movements are 8. The conflicts between right turn traffics are 4, and between left turn and merging traffic is 4. The conflicts created by pedestrians will be 8 taking into account all the four approaches. Diverging traffic also produces about 4 conflicts. Therefore, a typical four legged intersection has about 32 different types of conflicts. This is shown in figure 4.3.

The essence of the intersection control is to resolve these conflicts at the intersection for the safe and efficient movement of both vehicular traffic and pedestrians.

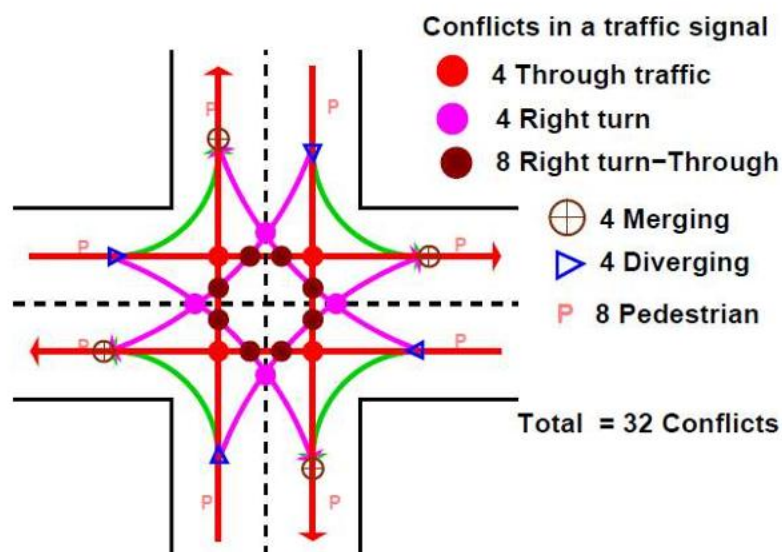


Figure 4.3: Conflicts at an intersection

4.4 Types of Intersections

Road intersections can be grouped in the following two categories:

Unit-IV Intersection Design

1. At-grade intersections
2. Grade separated

4.4.1 At-grade intersections

This is the most common type of intersection and it indicates the road junctions at the same elevation. The traffic movement like merging, diverging, crossing and weaving are carried out at the same level.

Following are the *three* types of at-grade intersections:

1. All-paved or unchannelized intersection
2. Channelized intersections
3. Roundabouts or rotaries

All-paved or unchannelized intersections: The unchannelized intersections are paved for the whole area and as such, there is no restriction to vehicles to use any part of the intersection area. These intersections are of the lowest order in the sense that traffic operations become complex in nature and for their smooth functioning they have to be controlled by traffic police or signals. Figure 4.4 shows the common varieties of all-paved intersections.

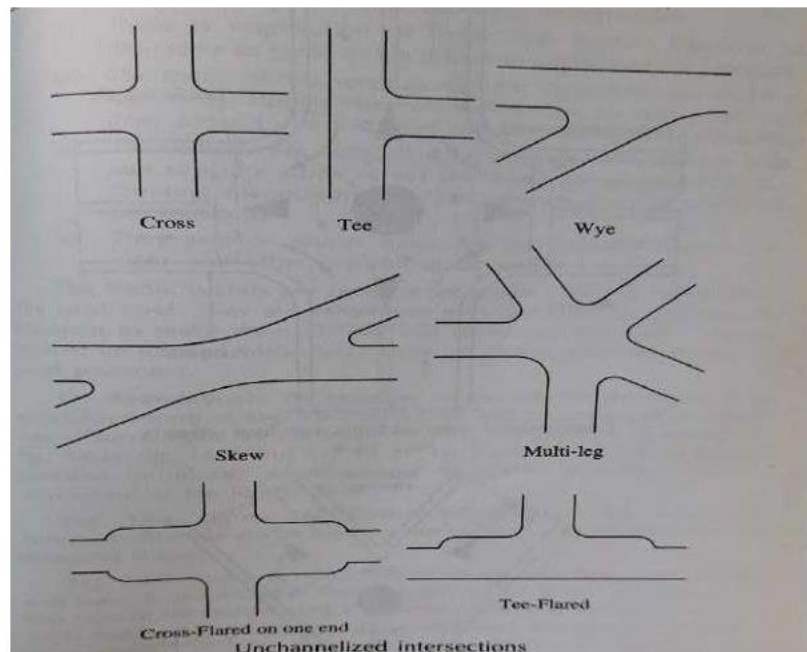


Figure 4.4 Unchannelized Intersections

Channelized intersections: In order to handle large volumes of traffic at the intersection, it becomes necessary to introduce traffic islands to channelize that turning traffic. The space at a road junction over which traffic is not allowed to make move is known as a *traffic island*. Figure 4.5 shows the arrangement of traffic islands for road junction with the *four* streets and *three* streets.

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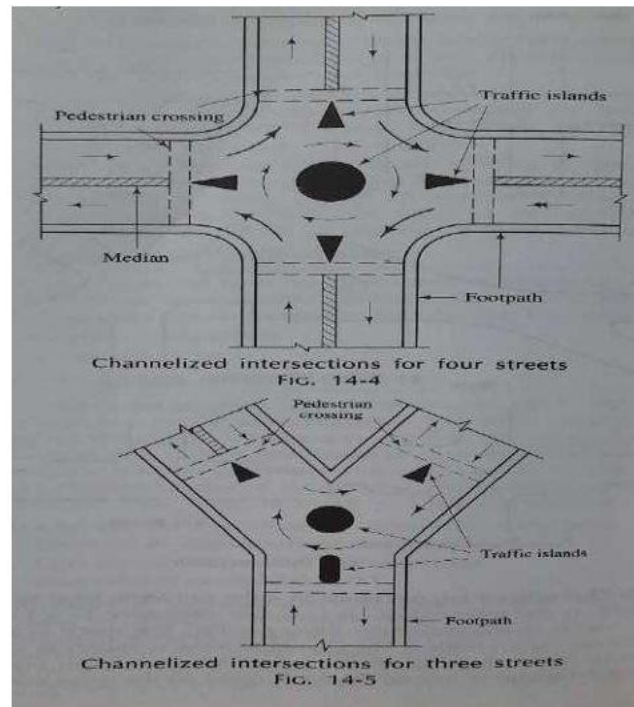


Figure 4.5: Traffic Islands for Road Junctions

The arrangement at road junction in the form of traffic islands serves the following five objectives:

1. The efficiency of traffic handling is increased.
2. There is segregation of traffic into proper channels as they serve as guide to the drivers in negotiating the junctions.
3. The traffic islands serve as shelters for pedestrians crossing the roads. Hence, they are referred to as *refugee islands*. They prove to be a special boon for the aged and infants.
4. The vehicles are compelled to move in one direction only and separate strips across the roads are provided for the crossing of pedestrians. Thus, the chance of accidents are reduced.
5. They provide proper place for the installation of traffic signs and other controlling devices.

Channelizing Devices

A channelizing device can be defined as any structure which helps in providing Channelization. These can be wide raised medians, non-traversable road islands, traversable raised curbs or even flush channelizing devices.

1. Wide Raised Medians

In this form of channelizing device, a raised wide separator is constructed between the two opposing lanes. Figure 4.6 shows a typical wide raised median on a freeway.

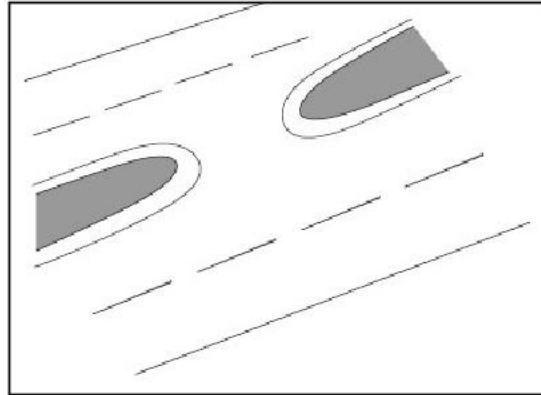


Figure 4.6: Wide Raised Median

2. Non- traversable Raised Islands

In this type of device, a narrower and a higher median than the traversable island is constructed between the opposing lanes. Due to the height, most of the vehicles are not able to cross the median, and hence the name. Figure 4.7 shows a non-traversable raised island constructed on a roadway.

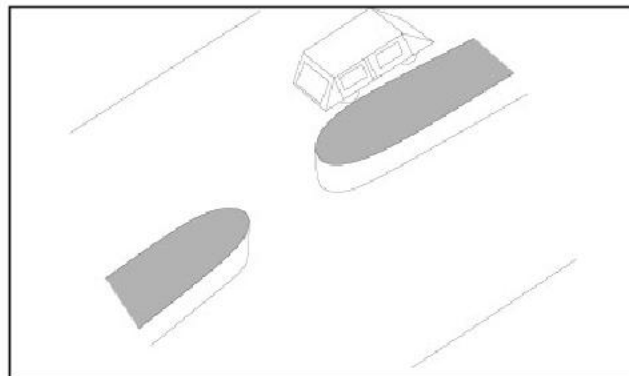
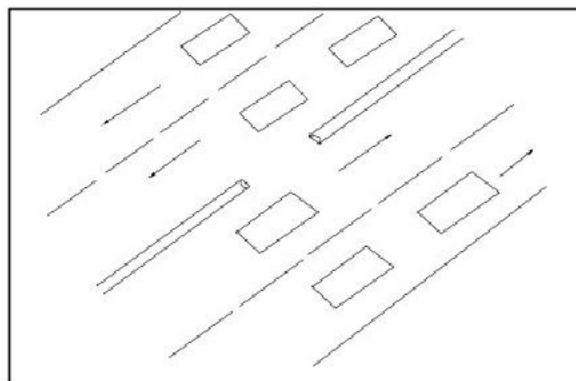


Figure 4.7: Non-traversable Raised Island

3. Traversable Raised Curb Systems

In this device, a narrow and mountable type of raised curb is constructed to separate the traffic moving in the opposing lanes. This class of channelizing device is the narrowest, and therefore the easiest to fit in a wide range of roadway cross-section widths. Figures 4.8 and 4.9 shows traversable raised curbs with and without vertical panels.



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Figure 4.8: Traversable Raised Curb System (Without vertical panels)

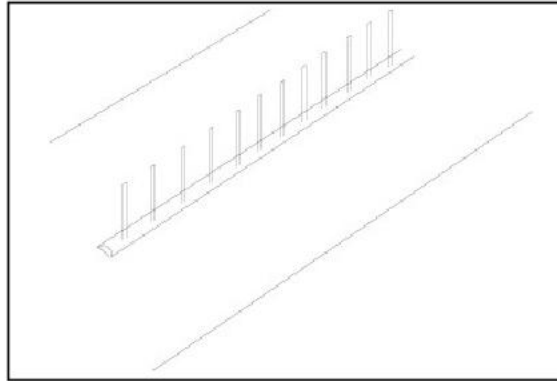


Figure 4.9: Traversable Raised Curb System (With vertical panels)

4. Flush Channelization

In this type of Channelization, a variety of treatments, including raising them above the pavement just slightly (2 to 5 cm); the application of pavement markings and other types of contrasting surfaces etc., are possible. The area seen flushed with the road surface in Figure 4.10 is the flush island.

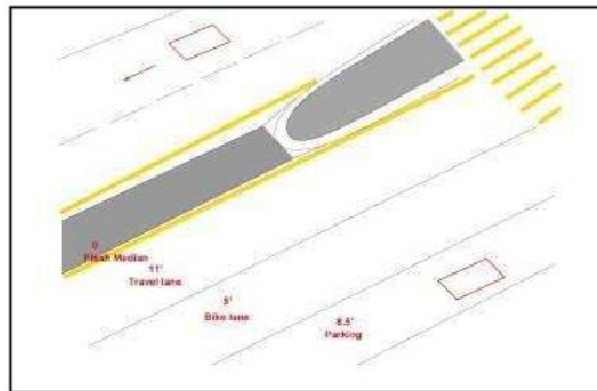


Figure 4.10 Flush Island providing Channelization objectives

Roundabouts or rotaries: A roundabout or rotary is a specialized form of at grade intersection and it consists of a central island surrounded by the roads forming the junction as shown in the figure 4.11. The provision of a roundabout junction introduces uni-directional (clockwise direction) circular movement at the intersection.

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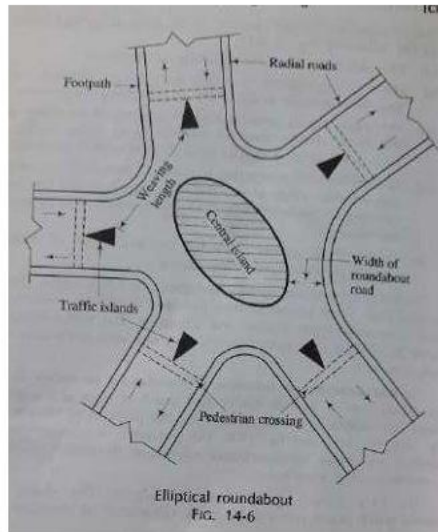


Figure 4.11: Roundabout

Following are the advantages of roundabouts:

1. It provides a simple solution for a road junction where more than four roads meet.
2. There is a little loss in traffic capacity as there is continuous flow of traffic through the roundabout. A well designed rotary can accommodate a traffic of about 3000 vehicles per hour and it also enables radial streets to carry traffic to their full capacity.
3. There is smooth and orderly flow of traffic with a little delay.
4. The vehicles as they approach the roundabouts are merging and weaving in the traffic stream. It thus provides a higher degree of safety and hence, there are less chances of serious collision or accidents.
5. The wear and tear of vehicles using the roundabouts are reduced as stopping and starting of the vehicles are less frequent.
6. The maintenance cost is considerably reduced due to the fact that the rotary can be made a self-controlled traffic intersection avoiding the necessity of traffic police or signals.

Following are the disadvantages of roundabouts

1. If traffic is to be stopped from time to time for the crossing of pedestrians, the orderly movement of traffic is seriously upset.
2. It becomes congested and hazardous under fog conditions.
3. It may confuse the drivers who are not familiar with such type of construction.
4. It requires large area of a flat land and hence, it cannot be adopted where land values are high.
5. It requires the installation of complicated traffic signs.
6. It proves to be unsuitable for high speed roads.

4.4.2 Grade separations

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A grade separated is the arrangement of taking one road over or under another by means of bridge. It is also known as a fly-over junction. There is no physical connection between the separated roadways in a grade separation.

The term *interchange* is used to indicate a grade separation having facility for traffic to pass from one intersection road to the other. Thus, connecting roads, known as *ramps* or *turning roadways* or provided to enable limited or full movement between the separated roads at an interchange. As shown in figure 4.12, the ramps can be classified in the following *three* categories:

- (1) Direct Ramp: The direct interchange ramp involves diverging rightside and merging from the right side
- (2) Semi-direct ramp: The semi-direct interchange ramp allows diverging on left, but merging is from the right side.
- (3) Indirect ramp: In the indirect interchange ramp the diverging is to the left and merging involved is also from the left side. This ramp proves to be easy, simple and less dangerous for the accidents. But the distance to be traversed for the transfer of route is more as compared to the previous two types.

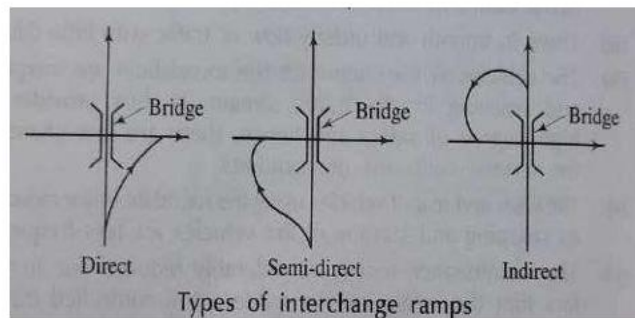


Figure 4.12 Types of interchange ramps

Classification of Grade Separated Intersection

As we discussed earlier, grade-separated intersections are provided to separate the traffic in the vertical grade. Different types of grade-separators are flyovers and interchange. Flyovers itself are subdivided into overpass and underpass. When two roads cross at a point, if the road having major traffic is elevated to a higher grade for further movement of traffic, then such structures are called overpass. Otherwise, if the major road is depressed to a lower level to cross another by means of an under bridge or tunnel, it is called under-pass. Interchange is a system where traffic between two or more roadways flows at different levels in the grade separated junctions. Common types of interchange include trumpet interchange, diamond interchange, and cloverleaf interchange.

1. **Trumpet interchange:** Trumpet interchange is a popular form of 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. A typical layout of trumpet interchange is shown in figure 4.13.

Unit-IV Intersection Design

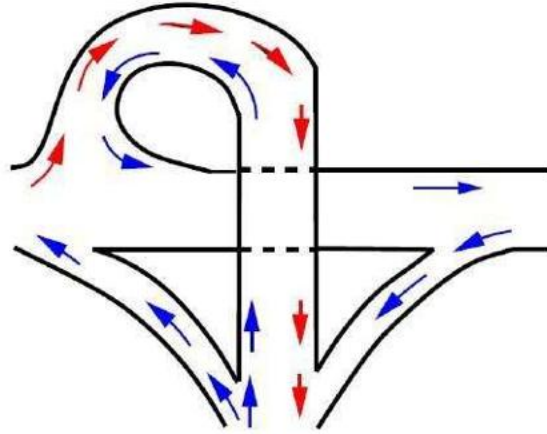


Figure 4.13 Trumpet Interchange

2. **Diamond interchange:** Diamond interchange is a popular form of four-leg interchange found in the urban locations where major and minor roads cross. The important feature of this interchange is that it can be designed even if the major road is relatively narrow. A typical layout of diamond interchange is shown in figure 4.14.

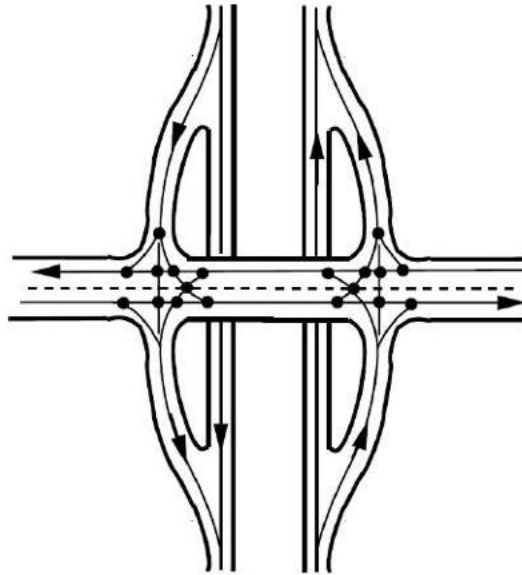


Figure 4.14: Diamond Interchange

3. **Cloverleaf interchange:** It is also a four leg interchange and is used when two highways of high volume and speed intersect each other with considerable turning movements. The main advantage of cloverleaf intersection is that it provides complete separation of traffic. The disadvantage is that large area of land is required. Therefore, cloverleaf interchanges are provided mainly in rural areas. A typical layout of this type of interchange is shown in figure 4.15.

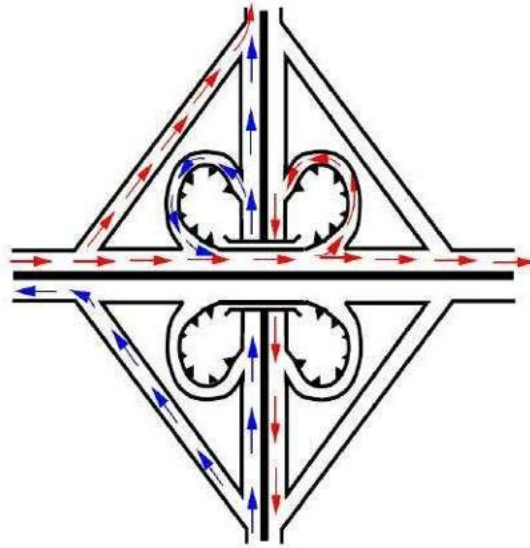


Figure 4.15: Cloverleaf interchange

Advantages of Grade Separator

The advantages of providing grade separator are as follows:

- 1) Grade separator provides maximum facility to the crossing traffic and avoids the accident while crossing.
- 2) They provide increased safety for turning traffic.
- 3) Grade separation is an essential part of controlled access highway like an expressway.
- 4) It is possible to adopt grade separation for all likely angles and layout of intersecting roads.
- 5) Stage construction of additional ramps is possible after the grade separation structure between main roads is construction.
- 6) Traffic capacity is increased.

Disadvantages of Grade Separator

The disadvantages of providing grade separator are as follows:

- 1) The construction cost is very high.
- 2) Construction of grade separator is costly, difficult and undesirable where there is limited right of way or where the topography is not favorable.
- 3) In flat or plain terrains, grade separation may introduce undesirable crests and sags in the vertical alignment.

4.5 Traffic Rotaries

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

Unit-IV Intersection Design

the rotary to the desired direction. The benefits, design principles, capacity of rotary etc. will be discussed in this chapter.

Guidelines for the selection

Because of the above limitation, rotaries are not suitable for every location. There are few guidelines that help in deciding the suitability of a rotary. They are listed below.

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

Traffic operations in a rotary

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

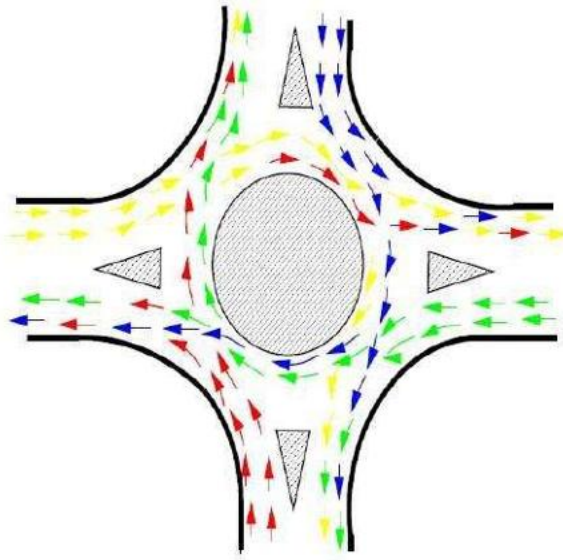


Figure 16 Traffic Operations in a rotary

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

Unit-IV Intersection Design

These movements are shown in figure 16. It can be observed that movements from each direction split into three; left, straight, and right turn.

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 formula. A typical rotary and the important design elements are shown in figure 17.

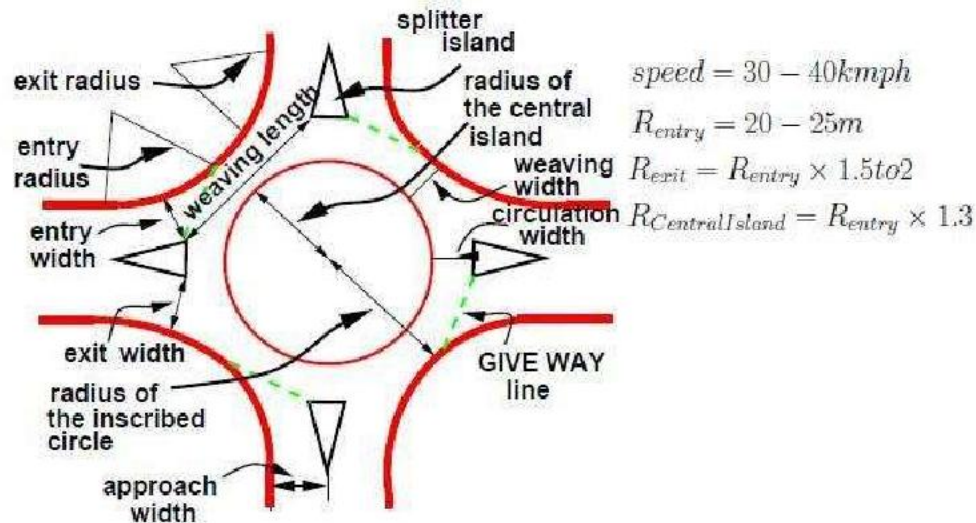


Figure 17 Design Elements of a rotary

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. 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 entry radius of about 20 and 25 meters is ideal for an urban and rural design respectively.

The exit radius should be higher than the entry 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.

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

Unit-IV Intersection Design

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,

$$W_{weaving} = \left(\frac{e_1 + e_2}{2} \right) + 3.5 \text{ m}$$

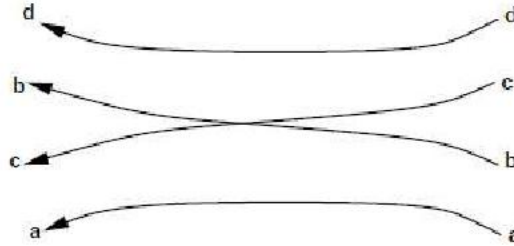


Figure 18 Weaving operations in a Rotary

where e_1 is the width of the carriageway at the entry and e_2 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.

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

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. Figure 18 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,

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

This capacity formula is valid only if the following conditions are satisfied.

1. Weaving width at the rotary is in between 6 and 18 meters.
2. 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.
3. The ratio of weaving width to weaving length of the roundabout is in between 0.12 and 0.4.

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4. The proportion of weaving traffic to non-weaving traffic in the rotary is in the range of 0.4 and 1.
5. The weaving length available at the intersection is in between 18 and 90 m.

4.6 Signalized intersection

The increasing of traffic volume at our intersection has been arise a problems like road accidents, conflicts and congestions. These problems can solved by providing an efficient traffic signal control at the intersection for continuous and efficient movement of vehicles through the intersection.

Definitions and notations

A number of definitions and notations need to be understood in signal design. They are discussed below:

- *Cycle*: A signal cycle is one complete rotation through all of the indications provided.
- *Cycle length*: Cycle length is the time in seconds that it takes a signal to complete one full cycle of indications. It indicates the time interval between the starting of green for one approach till the next time the green starts. It is denoted by C .
- *Interval*: Thus it indicates the change from one stage to another. There are two types of intervals - change interval and clearance interval. *Change interval* is also called the *yellow time* indicates the interval between the green and red signal indications for an approach. *Clearance interval* is also called *all red* is included after each yellow interval indicating a period during which all signal faces show red and is used for clearing off the vehicles in the intersection.
- *Green interval*: It is the green indication for a particular movement or set of movements and is denoted by G_i . This is the actual duration the green light of a traffic signal is turned on.
- *Red interval*: It is the red indication for a particular movement or set of movements and is denoted by R_i . This is the actual duration the red light of a traffic signal is turned on.
- *Phase*: A phase is the green interval plus the change and clearance intervals that follow it. Thus, during green interval, non-conflicting movements are assigned into each phase. It allows a set of movements to flow and safely halt the flow before the phase of another set of movements start.
- *Lost time*: It indicates the time during which the intersection is not effectively utilized for any movement. For example, when the signal for an approach turns from red to green, the driver of the vehicle which is in the front of the queue, will take some time to perceive the signal (usually called as reaction time) and some time will be lost here before he moves.
- *Saturation Flow (S)*: This indicates the number of vehicles passing an intersection with minimum headway during the whole of a 'green' period.

For new signal installations, the RRL recommends the use of the following formula for determining saturation flow –

$$S = 525 \text{ w passenger car units (PCU) per hour}$$

Unit-IV Intersection Design

Where,

S = Saturation flow

w = Width of approach road in metros.

This works out to approximately 160 PCU for every 0.3 m width of the road and is applicable for w ranging from 5.50m to 18m. For smaller widths, reduced values of S are recommended.

Methods for the Design of Traffic Signals:

The following methods are commonly used for the design of traffic signals:

- (a) Trial cycle method
- (b) Webster's minimum delay method
- (c) IRC method

Webster's method is a rational approach for signal design. The design is simple and is totally based on formulae's laid down by Webster. In this method, the total cycle of the signal is determined which forms a total least delay occurring at signal.

The steps in the design procedure are:

- 1) The saturation flow of traffic, $S_1, S_2, \dots$ on the approach roads are obtained from field studies by noting the number of vehicles in the stream of flow during the green aspects and the corresponding time intervals.
- 2) In the absence of data, the values are determined using RRL recommendations already given under the definition for 'saturation flow'. $S = 525 w$ passenger car units (PCU) per hour
- 3) The normal flows $q_1, q_2, \dots$ on the approach roads, during the design peak hour traffic are also obtained from field observations
- 4) The ratios $y_1 = q_1/S_1, y_2 = q_2/S_2, \dots$ are calculated.
- 5) In the case of mixed traffic, all the q - and S -values are to be obtained after converting to equivalent PCU values.
- 6) The optimum signal cycle, C_0 , is given by

$$C_o = \frac{1.5L + 5}{1 - Y}$$

Here, L = total lost time per cycle in seconds = $nl + R$

Where n is the number of phases based on traffic signal installations on the approach road and l is the lost time for the phase (as defined in 'lost time' earlier) and R is the all-red time in seconds.

If $Y = y_1 + y_2 + \dots$

Then $G_1 = \frac{y_1}{Y} (C_o - L)$

$$G_2 = \frac{y_2}{Y} (C_o - L)$$

And so on

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Providing suitable amber times, the times of red, green and amber aspects are obtained.

If there are pedestrian signals, the 'Walk' and 'Don't Walk' times can also be calculated based on the red aspect times on the particular roads and extra times for starting delays. The results may be summarized in the form of a timing diagram or a phasing diagram.

1. The width of a carriage way approaching an intersection is given as 15 m. The entry and exit width at the rotary is 10 m. The traffic approaching the intersection from the four sides is shown in the figure below. Find the capacity of the rotary using the given data.

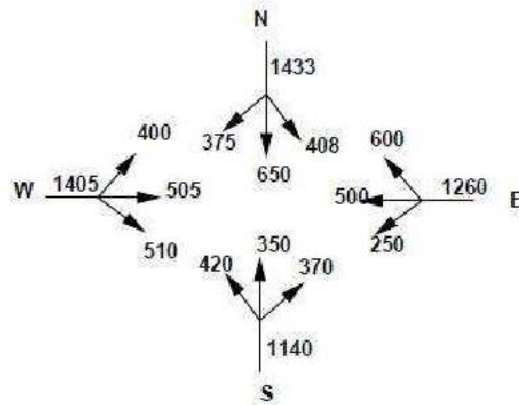


Figure: Traffic approaching the rotary

Solution

The traffic from the four approaches negotiating through the roundabout is illustrated in figure

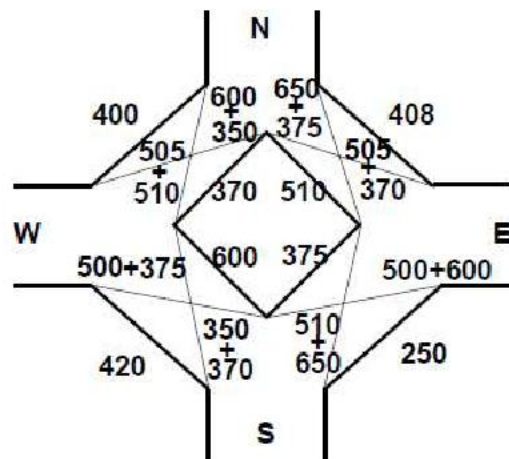


Figure: Traffic negotiating a rotary

- Weaving width is calculated as, $w = \left[\frac{e_1 + e_2}{2} \right] + 3.5 = 13.5m$
- Weaving length, l is calculated as, $l = 4 \times w = 54m$
- The proportion of weaving traffic to the non-weaving traffic in all the four approaches is found out first.
- It is clear from equation, that the highest proportion of weaving traffic to non-weaving traffic will give the minimum capacity. Let the proportion of weaving traffic to the non-

weaving traffic in West-North direction be denoted as p_{WN} , in North-East direction as p_{NE} , in the East-South direction as p_{ES} , and finally in the South-West direction as p_{SW} .

- The weaving traffic movements in the East-South direction is shown in figure 6. Then using equation,

$$p_{ES} = \frac{510 + 650 + 500 + 600}{510 + 650 + 500 + 600 + 250 + 375} = \frac{2260}{2885} = 0.783$$

$$p_{WN} = \frac{505 + 510 + 350 + 600}{505 + 510 + 350 + 600 + 400 + 370} = \frac{1965}{2735} = 0.718$$

$$p_{NE} = \frac{650 + 375 + 505 + 370}{650 + 375 + 505 + 370 + 510 + 408} = \frac{1900}{2818} = 0.674$$

$$p_{SW} = \frac{350 + 370 + 500 + 375}{350 + 370 + 500 + 375 + 420 + 600} = \frac{1595}{2615} = 0.6099$$

- Thus the proportion of weaving traffic to non-weaving traffic is highest in the East-South direction.

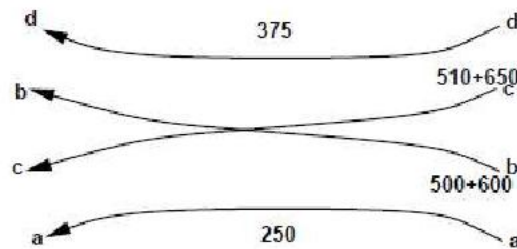


Figure: Traffic weaving in East-South direction

Therefore, the capacity of the rotary will be capacity of this weaving section. From equation,

$$Q_{ES} = \frac{280 \times 13.5 \left[1 + \frac{10}{13.5}\right] \left[1 - \frac{0.783}{3}\right]}{1 + \frac{13.5}{54}} = 2161.164 \text{ veh/hr}$$

- The width of approaches for a rotary intersection is 12 m. The entry and exit width at the rotary is 10 m. Table below gives the traffic from the four approaches, traversing the intersection. Find the capacity of the rotary.

Approach	Left turn	Straight	Right turn
North	400	700	300
South	350	370	420
East	200	450	550

West	350	500	520
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Solution

The traffic from the four approaches negotiating through the roundabout is illustrated in figure

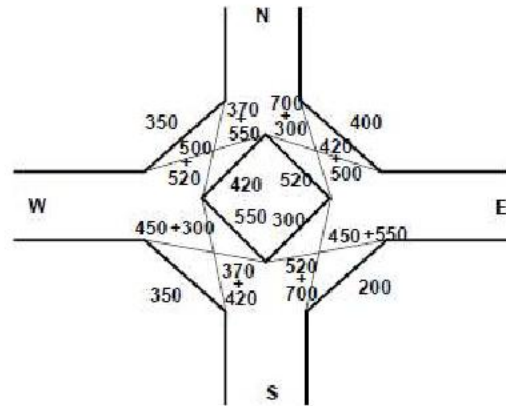


Figure: Traffic negotiating a rotary

- Weaving width is calculated as, $w = \left[\frac{e_1 + e_2}{2} \right] + 3.5 = 13.5m$
- Weaving length, l is calculated as, $l = 4 \times w = 54m$
- The proportion of weaving traffic to the non-weaving traffic in all the four approaches is found out first.
- It is clear from equation, that the highest proportion of weaving traffic to non-weaving traffic will give the minimum capacity. Let the proportion of weaving traffic to the non-weaving traffic in West-North direction be denoted as p_{WN} , in North-East direction as p_{NE} , in the East-South direction as p_{ES} , and finally in the South-West direction as p_{SW} .

$$p_{ES} = \frac{450 + 550 + 700 + 520}{200 + 450 + 550 + 700 + 520 + 300} = \frac{2220}{2720} = 0.816$$

$$p_{WN} = \frac{370 + 550 + 500 + 520}{350 + 370 + 550 + 500 + 520 + 420} = \frac{1740}{2510} = 0.690$$

$$p_{NE} = \frac{420 + 500 + 700 + 300}{520 + 400 + 420 + 500 + 700 + 300} = \frac{1920}{2840} = 0.676$$

$$p_{SW} = \frac{450 + 300 + 370 + 420}{550 + 450 + 370 + 420 + 350} = \frac{1540}{2540} = 0.630$$

- Thus the proportion of weaving traffic to non-weaving traffic is highest in the East-South direction.

- Therefore, the capacity of the rotary will be the capacity of this weaving section. From equation,

$$Q_{ES} = \frac{280 \times 13.5 \left[1 + \frac{10}{13.5}\right] \left[1 - \frac{0.816}{3}\right]}{1 + \frac{13.5}{54}} = 380.56 \text{ veh/hr}$$

3. The average normal flow of traffic on cross roads A and B during design periods are 400 and 250 PCU/hr; the saturation flow values on these roads are estimated as 1250 and 1000 PCU/hr respectively. The all-red time required for pedestrian crossings is 12 sec. Design two phase traffic signal with pedestrian crossing by Webster's method.

Solution

Given: normal flow on roads A & B: $q_a = 400 \text{ PCU/hr}$ and $q_b = 250 \text{ PCU/hr}$

Saturation flow, $S_a = 1250 \text{ PCU/hr}$ and $S_b = 1000 \text{ PCU/hr}$;

All-red time, $R = 12 \text{ sec}$

Number of Phases, $n = 2$

$$Y_a = \frac{q_a}{S_a} = \frac{400}{1250} = 0.32$$

$$Y_b = \frac{q_b}{S_b} = \frac{250}{1000} = 0.25$$

$$Y = y_a + y_b = 0.32 + 0.25 = 0.57$$

Total Lost Time, $L = 2n + R = 2 \times 2 + 12 = 16 \text{ sec}$

Optimum Cycle time,

$$C_o = \frac{1.5L + 5}{1 - Y} = \frac{1.5 \times 16 + 5}{1 - 0.57} = \frac{29}{0.43} = 67.4 \cong 67.5 \text{ sec}$$

$$G_a = \frac{y_a}{Y} (C_o - L) = \frac{0.32}{0.57} (67.5 - 16) = 29 \text{ sec}$$

$$G_b = \frac{y_b}{Y} (C_o - L) = \frac{0.25}{0.57} (67.5 - 16) = 22.5 \text{ sec}$$

Provide an all-red time, R for pedestrian crossing = 12 Sec

Providing Amber times of 2.0 sec each for clearance, total cycle time

$$= 29 + 22.5 + 12 + 2 + 2 = 67.5 \text{ sec}$$

4. The average normal flow of traffic on cross roads 1 and 2 during design periods are 440 and 280 PCU/hr; the saturation flow values on these roads are estimated as 1300 and 1100 PCU/hr respectively. The all-red time required for pedestrian crossings is 12 sec. Design two phase traffic signal with pedestrian crossing by Webster's method.

Solution

Given: normal flow on roads A & B: $q_1 = 440 \text{ PCU/hr}$ and $q_2 = 280 \text{ PCU/hr}$

Saturation flow, $S_1 = 1300 \text{ PCU/hr}$ and $S_2 = 1100 \text{ PCU/hr}$;

All-red time, $R = 12 \text{ sec}$

Number of Phases, $n = 2$

$$Y_1 = \frac{q_1}{S_1} = \frac{440}{1300} =$$

$$Y_2 = \frac{q_2}{S_2} = \frac{280}{1100} =$$

$$Y = y_1 + y_2 =$$

Total Lost Time, $L = 2n + R = 2 \times 2 + 12 = 16 \text{ sec}$

Optimum Cycle time,

$$C_o = \frac{1.5L + 5}{1 - Y} =$$

$$G_1 = \frac{y_1}{Y} (C_o - L) =$$

$$G_2 = \frac{y_2}{Y} (C_o - L) =$$

Provide an all-red time, R for pedestrian crossing = 12 Sec

Providing Amber times of 2.0 sec each for clearance, total cycle time

=

UNIT-I

PAVEMENT DESIGN

A highway pavement is a structure consisting of superimposed layers of processed materials above the natural soil sub-grade, whose primary function is to distribute the applied vehicle loads to the sub-grade. The pavement structure should be able to provide a surface of acceptable riding quality, adequate skid resistance, favorable light reflecting characteristics, and low noise pollution. The ultimate aim is to ensure that the transmitted stresses due to wheel load are sufficiently reduced, so that they will not exceed bearing capacity of the sub-grade. Two types of pavements are generally recognized as serving this purpose, namely flexible pavements and rigid pavements. This chapter gives an overview of pavement types, layers, and their functions, and pavement failures. Improper design of pavements leads to early failure of pavements affecting the riding quality.

Requirements of a pavement

An ideal pavement should meet the following requirements:

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

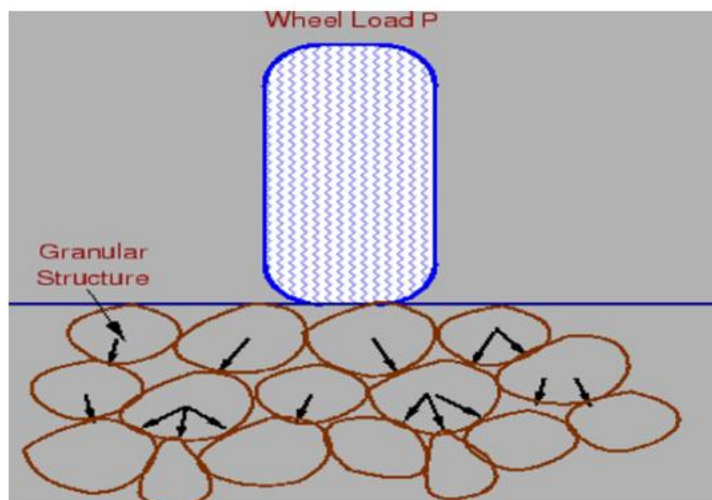
Types of pavements

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

Flexible pavements

- Flexible pavements will transmit wheel load stresses to the lower layers by grain-to-grain transfer through the points of contact in the granular structure.
- The wheel load acting on the pavement will be distributed to a wider area, and the stress decreases with the depth. Taking advantage of these stress distribution characteristic, flexible pavements normally has many layers.

- Hence, the design of flexible pavement uses the concept of layered system. Based on this, flexible pavement may be constructed in a number of layers and the top layer has to be of best quality to sustain maximum compressive stress, in addition to wear and tear. The lower layers will experience lesser magnitude of stress and low quality material can be used. Flexible pavements are constructed using bituminous materials.
- These can be either in the form of surface treatments (such as bituminous surface treatments generally found on low volume roads) or, asphalt concrete surface courses (generally used on high volume roads such as national highways).
- Flexible pavement layers reflect the deformation of the lower layers on to the surface layer (e.g., if there is any undulation in sub-grade then it will be transferred to the surface layer). In the case of flexible pavement, the design is based on overall performance of flexible pavement, and the stresses produced should be kept well below the allowable. Stresses of each pavement layer.



FUNCTIONS OF A FLEXIBLE PAVEMENT LAYERS

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

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

Prime Coat: It is an application of low viscous cutback bitumen to an absorbent surface like granular bases on which binder layer is placed.

It provides bonding between two layers. Unlike tack coat, prime coat penetrates into the layer below, plugs the voids, and forms a water tight surface.

SURFACE COURSE

- It is the layer directly in contact with traffic loads and generally contains superior quality materials. They are usually constructed with dense graded asphalt concrete (AC).

The functions and requirements of this layer are:

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

BINDER COURSE

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

BASE COURSE

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

COMPOSITION OF FLEXIBLE PAVEMENTS



SUB - BASE COURSE

- It is the layer of material beneath the base course and the primary functions are to provide structural support, improve drainage and reduce the intrusion of fines from the sub-grade in the pavement structure

- If the base course is open graded, then the sub-base course with more fines can serve as a filler between sub-grade and the base course
- A sub-base course is not always needed or used.
- For example, a pavement constructed over a high quality, stiff sub-grade may not need the additional features offered by a sub-base course. In such situations, sub-base course may not be provided

SUB – GRADE: The top soil or sub-grade is a layer of natural soil prepared to receive the stresses from the layers above.

Types of Flexible Pavements

The following types of construction have been used in flexible pavement:

- Conventional layered flexible pavement,
- Full - depth asphalt pavement, and
- Contained rock asphalt mat (CRAM).

Conventional flexible pavements are layered systems with high quality expensive materials are placed in the top where stresses are high, and low quality cheap materials are placed in lower layers.

Full - depth asphalt pavements are constructed by placing bituminous layers directly on the soil sub-grade. This is more suitable when there is high traffic and local materials are not available.

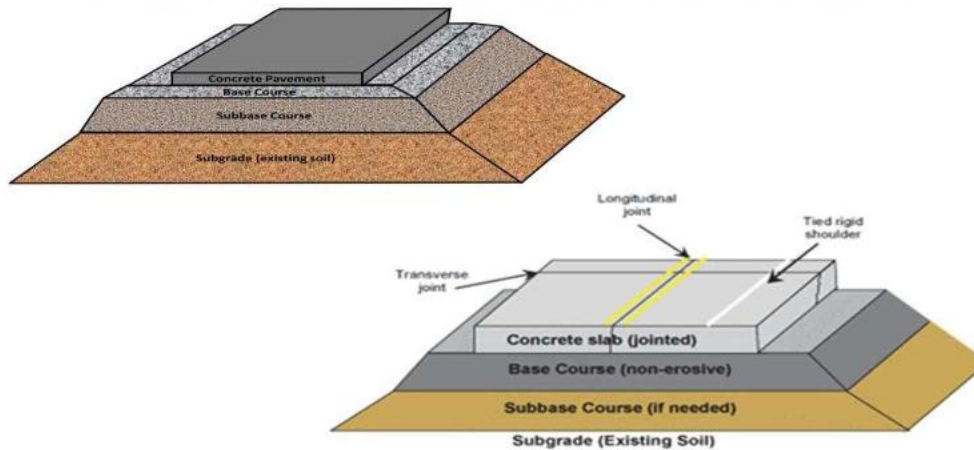
Contained rock asphalt mats are constructed by placing dense/open graded aggregate layers in between two asphalt layers. Modified dense graded asphalt concrete is placed above the sub-grade will significantly reduce the vertical compressive strain on soil sub-grade and protect from surface water.

Rigid pavements

- Rigid pavements have sufficient flexural strength to transmit the wheel load stresses to a wider area below. Compared to flexible pavement, rigid pavements are placed either directly on the prepared sub-grade or on a single layer of granular or stabilized material. Since there is only one layer of material between the concrete and the sub-grade, this layer can be called as base or sub-base course.
- In rigid pavement, load is distributed by the slab action, and the pavement behaves like an elastic plate resting on a viscous medium. Rigid pavements are constructed by Portland cement concrete (PCC) and should be analyzed by plate theory instead of layer theory, assuming an elastic plate resting on viscous foundation.
- Plate theory is a simplified version of layer theory that assumes the concrete slab as a medium thick plate which is plane before loading and to

remain plane after loading. Bending of the slab due to wheel load and temperature variation and the resulting tensile and flexural stress.

COMPOSITION OF RIGID PAVEMENTS



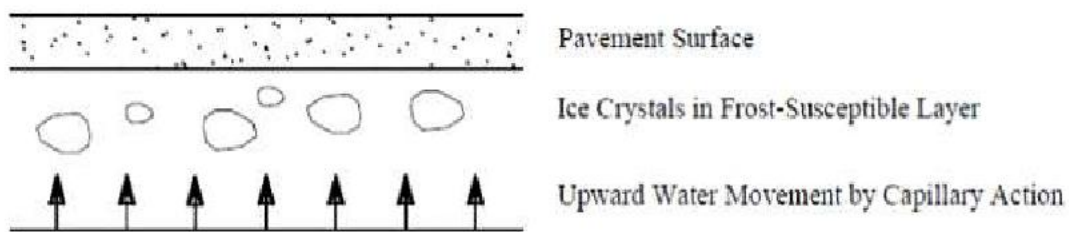
FUNCTIONS OF A FLEXIBLE PAVEMENT LAYERS

CONCRETE SLAB / SURFACE COURSE: The concrete slab is the top most layer of rigid pavement which is in direct contact with the vehicular loads. This is also called as surface course. It is water resistant and prevents the water infiltration into the base course. It offers friction to the vehicles to provide skid resistance. The thickness of concrete slab is kept between 150 mm to 300 mm.

GRANULAR SUB BASE OR STABILIZED SUB BASE COURSE It is the third layer from the top and is in contact with the sub grade soil and base course. It is constructed by using low quality aggregates than the base course but they should be better quality than sub grade. Generally sub base course is not required when the traffic loading is light but when the loading exceeds 100000 pounds / 45000 Kgs it should be constructed. Its primary function is to provide support for the top layers and it also serves as frost action controller and prevents the intrusion of fines from sub grade to top layers. The drainage facility will also improve when there is a sub base course.

FROST PROTECTION LAYER: If the soil contains high ground water table, during low temperatures the water will freeze and frost heave will formed under the sub grade which will cause the pavement to rise because of non-uniform formation of ice crystals.

- If the soil contains high ground water table, during low temperatures the water will freeze and frost heave will formed under the sub grade which will cause the pavement to rise because of non-uniform formation of ice crystals.
- Similarly, when the ice melts the pavement will penetrate into the sub grade when load comes on it. Generally a good base course and sub base course themselves acts as frost protection layers.



COMPARISON OF FLEXIBLE PAVEMENT & RIGID PAVEMENT:

FLEXIBLE PAVEMENT	RIGID PAVEMENT
Have low flexural strength	Have more flexural strength
Load is transferred by grain to grain contact	No such phenomenon of grain to grain load transfer exists
Surfacing cannot be laid directly on the sub grade but a sub base is needed	Surfacing can be directly laid on the sub grade
No thermal stresses	Thermal stresses are induced
Expansion joints are not needed	Expansion joints are needed
Design life 10-15 years	Design life 20-30 years
Initial cost of construction is low	Initial cost of construction is high
Maintenance cost is high	Less maintenance cost
Road can be used for traffic within 24 hours	Road cannot be used until 14 days of curing
Damaged by Oils and Certain Chemicals	Cannot be damaged by oils and chemicals.

