B. E. CIVIL ENGINEERING
Choice Based Credit System (CBCS) and Outcome Based Education (OBE)
SEMESTER - V

HIGHWAY ENGINEERING

Course Code	**18CV56**	CIE Marks	40
Teaching Hours/Week(L:T:P)	(3:0:0)	SEE Marks	60
Credits	03	Exam Hours	03

Course Learning Objectives: This course will enable students to;

1. Gain knowledge of different modes of transportation systems, history, development of highways and the organizations associated with research and development of the same in INDIA.
2. Understand Highway planning and development considering the essential criteria's (engineering and financial aspects, regulations and policies, socio economic impact).
3. Get insight to different aspects of geometric elements and train them to design geometric elements of a highway network.
4. Understand pavement and its components, pavement construction activities and its requirements.
5. Gain the skills of evaluating the highway economics by B/C, NPV, IRR methods and also introduce the students to highway financing concepts.

Module -1

Principles of Transportation Engineering: Importance of transportation, Different modes of transportation and comparison, Characteristics of road transport Jayakar committee recommendations, and implementation – Central Road Fund, Indian Roads Congress, Central Road Research Institute.

Highway Development and Planning: Road types and classification, road patterns, planning surveys, master plan – saturation system of road planning, phasing road development in India, problems on best alignment among alternate proposals Salient Features of 3rd and 4thtwenty year road development plans and Policies, Present scenario of road development in India (NHDP & PMGSY) and in Karnataka (KSHIP & KRDCL) Road development plan - vision 2021.

Highway Alignment and Surveys: Ideal Alignment, Factors affecting the alignment, Engineering surveys-Map study, Reconnaissance, Preliminary and Final location & detailed survey, Reports and drawings for new and re-aligned projects.

Module -2

Highway Geometric Design of horizontal alignment elements: Cross sectional elements–width, surface, camber, Sight distances–SSD, OSD, ISD, HSD, Radius of curve, Transition curve, Design of horizontal and vertical alignment–curves, super-elevation, widening, gradients, summit and valley curves.

Module -3

Pavement Materials: Sub grade soil - desirable properties-HRB soil classification-determination of CBR and modulus of sub grade reaction with Problems Aggregates- Desirable properties and tests, Bituminous materials- Explanation on Tar, bitumen, cutback and emulsion-tests on bituminous material Pavement Design: Pavement types, component parts of flexible and rigid pavements and their functions, ESWL and its determination (Graphical method only)-Examples.

Module -4

Pavement Construction: Design of soil aggregate mixes by Rothfuch's method. Uses and properties of bituminous mixes and cement concrete in pavement construction. Earthwork; cutting and Filling, Preparation of subgrade, Specification and construction of i) Granular Sub base, ii) WBM Base iii) WMM base,iv) Bituminous Macadam v) Dense Bituminous Macadam vi) Bituminous Concrete,vii) Dry Lean Concrete sub base and PQC viii) concrete roads.

Module -5

Highway Drainage: Significance and requirements, Surface drainage system and design-Examples, sub surface drainage system, design of filter materials, Types of cross drainage structures, their choice and location.

Highway Economics: Highway user benefits, VOC using charts only-Examples, Economic analysis - annual cost method-Benefit Cost Ratio method-NPV-IRR methods- Examples, Highway financing-BOT-BOOT concepts.

Course Outcomes: After studying this course, students will be able to:

1. Acquire the capability of proposing a new alignment or re-alignment of existing roads, conduct necessary field investigation for generation of required data.
2. Evaluate the engineering properties of the materials and suggest the suitability of the same for pavement construction.
3. Design road geometrics, structural components of pavement and drainage.
4. Evaluate the highway economics by few select methods and also will have a basic knowledge of various highway financing concepts.

Question paper pattern:

- The question paper will have ten full questions carrying equal marks.
- Each full question will be for 20 marks.
- There will be two full questions (with a maximum of four sub- questions) from each module.
- Each full question will have sub- question covering all the topics under a module.
- The students will have to answer five full questions, selecting one full question from each module.

Textbooks:

1. S K Khanna and C E G Justo, "Highway Engineering", Nem Chand Bros, Roorkee.
2. L R Kadiyali, "Highway Engineering", Khanna Publishers, New Delhi.
3. R Srinivasa Kumar, "Highway Engineering", University Press.
4. K. P.Subramanium, "Transportation Engineering", SciTech Publications, Chennai.

Reference Books:

1. Relevant IRC Codes.
2. Specifications for Roads and Bridges-MoR T&H, IRC, New Delhi.
3. C. JotinKhisty, B. Kentlal, "Transportation Engineering", PHI Learning Pvt. Ltd. New Delhi.

HIGHWAY ENGINEERING (15CV63)

As per Choice Based Credit System (CBCS) Scheme

MODULE – 1

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MODULE – 2

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Highway Geometric Design: Cross sectional elements–width, surface, camber, Sight distances–SSD, OSD, ISD, HSD, Design of horizontal and vertical alignment–curves, super-elevation, widening, gradients, summit and valley curves

MODULE – 3

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1) Granular Sub base,
2) WBM Base,
3) WMM base,
4) Bituminous Macadam,
5) Dense Bituminous Macadam
6) Bituminous Concrete,
7) Dry Lean Concrete sub base and PQC
8) Concrete Roads

MODULE – 5

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Highway Economics: Highway user benefits, VOC using charts only-Examples,
Economic analysis - annual cost method-Benefit Cost Ratio method-NPV-IRR
methods- Examples, Highway financing-BOT-BOOT concepts

MODULE – 1

PRINCIPLES OF TRANSPORTATION ENGINEERING

INTRODUCTION

Basic Definition

A facility consisting of the means and equipment necessary for the movement of passengers or goods. At its most basic, the term **"Transportation System"** is used to refer to the equipment and logistics of transporting passengers and goods.

Importance of Transportation

The evolution and advancements in transportation facilities have been closely linked with the development of human beings throughout the history of the world.

Role of Transportation

Transportation plays a vital role in economic development of any region of any country, since every commodity produced, whether it may be agricultural or industrial products they need to be transported at various stages from production to distribution. At production stage for carrying raw materials and at distribution stage for transportation from farms and factories to marketing centers to retailers to consumers.

Inadequate transportation facilities retard the process of socio-economic and cultural development. Development of transportation facilities in a country indicates its economic growth and progress in social development.

The main objective of a good transportation system is to provide a safe, economical and efficient transportation facility for passengers and goods.

Economic Activity and Transport

These are the processes in which the products are utilized to satisfy human needs. Two important factors well known in economic activity are

1) Production or supply
2) Consumption for human needs or demands

Social Effects of Transportation

The progress of a nation depends on transportation facilities. The population usually settles along the transportation routes such as road sides, river shores and railway stations. However, in the present concept of road network planning the above said kind of ribbon development is discouraged for the sake of high speed travel and safety. Attempts are being made to decentralize the population away from main transportation routes.

To avoid congestion on major cities, suburbs and satellite towns are being developed and are linked to the major cities with mass rapid transit system.

The various social effects of transportation are

a) Sectionalism and transportation
b) Concentration of population in urban area
c) Aspect of safety, law and order

a) Sectionalism and Transportation

1) Improved transportation has important implication in reducing sectionalism within the country and also with other countries in the world
2) The living conditions and facilities of under developed colonies and tribes get improved since the distances are apparently reduced with reduction in travel time.
3) Frequent travel to the other parts of the country and outside the country tend to increase knowledge of the people by learning from other sections of society which results in improved trade and cultural exchanges.
4) International understanding for the better peace and order also improves with efficient network of transportation.

b) Concentration of Population in Urban Areas

1) Improved transportation facilities bring prosperity to the urban population
2) The employment opportunities, prosperity and superior facilities for education, medical care etc., are available in urban areas attract the population from other areas leading to increased economic activities
3) Adequate mass transportation facilities are needed to cater for the internal movements for daily movements and other social needs
4) Effective rapid transportation facilities are needed for suburban and intercity long-distance travel

5) Inadequate transportation facilities lead to concentration of population in cities which often results in congestion and related issues.
6) If adequate facilities are provided people tend to prefer to reside at localities away from urban centers.

c) Aspect of safety, Law and order

1) Transportation facilities are required for rushing aid to areas affected by an emergency.
2) To maintain law and order and defend the territory of the country against external aggression and to guard borders with foreign territory transportation facilities are needed.
3) Sometimes defense needs alone are a sufficient reason to develop transportation needs which may not have any social and economic benefits.

Role of Transportation for the Development of rural areas in India

About 70% of population in India are living in rural areas. Therefore, development in urban centers alone do not indicate overall development of the country. Only with the improvements in transportation facilities in rural areas, there could be faster development of these areas, resulting in overall development of country.

Impacts of rural roads connectivity from rural road development in India

1) Improvements in transportation services leads to improved access to market centers for the rural producers, better availability of farm inputs at reduced prices.
2) Diversification of agricultural produce with improved market access promotes shift in favor of cash crops and commercialization of agricultural activities.
3) Diversification of livelihood opportunities with better connectivity enhances employment opportunity with better connectivity enhances employment opportunities in non-agricultural sectors.
4) Improved services with improved road connectivity, inter-alia, enhances access to education, health and financial services.
5) Increase in outreach due to improved rural roads facilities better availability of public services and functionaries in rural areas.

DIFFERENT MODES OF TRANSPORTATION

Transportation has developed along three basic modes of transport

a) Land
b) Water
c) Air

Land has given scope for development of transportation by road and rail transport. Water and air media have developed waterways and airways respectively. The roads or the highways not only include modern highway system but also includes the urban arterials, city streets, feeder roads and village roads catering for a wide variety of vehicles and pedestrians. Railways have been developed both for long distance travel and also urban travel. Waterways include transportation by oceans, rivers, canals and lakes for the movement of ships and boats. The airways help in faster transportation by aircrafts and carriers.

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:

a) Roadways or highways for road transportation
b) Railways for rail transportation
c) Waterways for water transportation
d) Airways for air transportation

ROADWAYS

The transportation by road is the only mode which could give maximum service to one and all. Road transport mode has the maximum flexibility for travel with reference to choice of the route, direction, time and speed of travel. This is only mode which caters for the movement of passengers and goods independently right from the place of origin up to the destination of any trip along the route. The other three modes (railways; water ways; airways) have to depend on transportation by road for the service to and from their respective terminals. Therefore, the roadway essentially serves as a feeder network. It is possible to provide door to door service by road transport. Ultimately, road network is therefore needed not only to serve as feeder system for other modes of transportation and to supplement them, but also to provide independent facility for road travel by a well-planned network of roads throughout the country

Advantages:

1) Flexibility: It offers complete freedom to the road users.
2) It requires relatively smaller investments and cheaper in construction with respect to other modes.
3) It serves the whole community alike the other modes.
4) For short distance travel, it saves time.
5) The road network is used by various types of vehicles.

Disadvantages:

1) Speed is related to accidents and more accidents results due to higher speed and is usually not suitable for long distance travel
2) Power required per tonne is more.

RAILWAYS

The concept of rail transportation is movement of multiple wagons or a train of wagons passenger's bogies on two parallel steel rails. The resistance to traction along the railway track for the movement of steel wheels is much lower than that along more uneven road surface for the movement of road vehicles with rubber tyres. The transportation along the railway track could be advantageous by railways between the stations both for the passengers and goods, particularly for longer distances. The energy requirement to haul unit load through unit distance by the railway is only a fraction (one fourth to one sixth) of the required by road. Hence, full advantage of this mode of transportation should be taken for the transportation of bulk goods along land where the railway facilities are available. The Indian railways is one of the world's largest Railway network in the world. It was introduced in 1853 and it is spread over 1,09,221 km covering 6906 stations.

Advantages:

1) Can transport heavy loads of goods at higher speed
2) Power required per tonne is less compared to roadways
3) Chances of accidents are less.

Disadvantages:

1) Entry and exist points are fixed
2) Requires controlling system and no freedom of movement
3) Establishment and maintenance cost is higher

WATERWAYS

Transportation by water offers minimum resistance to traction and therefore needs minimum energy to haul unit load through distance. The water transportation is the most energy efficient but it is the slowest among the four modes. The highest use of this mode is for bulk cargo of relatively low value. The transportation by water is possible between the ports on the sea routes or along the rivers or canals where inland transportation facilities are available.

Advantages:

1) Cheapest: Cost per tonne is lowest
2) Possess highest load carrying capacity
3) Leads to the development of the industries.

Disadvantages:

1) Slow in operation and consumes more time and Depends on whether condition
2) Chances of attack by other countries on naval ships are more.
3) Ocean tides affects the loading and unloading operation and the routes are circuitous.

AIRWAYS

The transportation by air is the fastest among the four modes. Air transport provides more comfortable and fast travel resulting in substantial saving in travel time for the passengers between the airports. The shipment of high value freight on long hauls is possible in the shortest time by air transport. Unlike other modes of transport, air transport allows continuous journey over the land and water, even across inaccessible places in between two airports.

For shorter hauls helicopters are used and they were developed for their landing and takeoff. Military aviation is also important to meet the defense needs of a country.

Advantages

1) It has highest speed.
2) Intercontinental travel is possible
3) Journey is continuous over land and water

Disadvantages

1) Highest operating cost (cost/tonne is more) and the load carrying capacity is lowest
2) Depends on whether condition
3) Should follow the flight rules.

CHARACTERISTICS OF ROAD TRANSPORTATION

It is accepted that the fact road transport is the nearest to the people. All classes of road vehicles consisting of both personal or public transport vehicles and also the pedestrians can make use of the roadway system. The passengers and goods have to be first transported by road before reaching a railway station or an airport. The far-flung border areas located in high altitude and difficult terrains of the country and the remote villages in the under developed villages could be served by the road network. Road network is very economical and convenient for short road trips and even some times for longer trips.

The characteristics are of roads are as follows

1) Roads are used by various types of road vehicles like passenger, goods vehicles and pedestrians. But the rail locomotives and wagons can only make use of the railway track. The ships and boats can make use of only the waterways and the aircraft's only the airports.
2) Road transport infrastructure requires the lowest initial investments in comparison to that for the infrastructure of other transportation modes. The cost of any class of road of road vehicle is much lower is much lower than that of other carriers like the railways, ships and aircrafts. The initial cost of construction and the cost of maintenance of roads is also lesser than those for railway tracks, harbors and airports.
3) Roads offer complete freedom to the roads to the road user to make use of the roadway facilities at any time convenient to them or to move the vehicle from a lane of the road to the adjoining one and from one road to another, according to the need and convenience.
4) It is possible to travel directly from the respective places of origin to the destination by road vehicles.
5) Speed of movement is directly related with the severity of accidents. The road safety decreases with the increasing running speed dispersion in the traffic stream. Road transport is prone to a high rate of accidents due to the flexibility of movements offer to the road users. However, in other modes of transport, in spite of various safety measures and strict controls in the movements, major accidents do occur even in the form of head on collisions and the accidents in these modes are more severe and disastrous.
6) Road transport is the only mode that offers the facilities to the whole section of society.

JAYAKAR COMMITTEE RECOMMENDATIONS AND IMPLEMENTATION

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 when not capable to 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

1) The road development in the country should be considered as a national interest as this has become beyond the capacity of provincial governments and local bodies.
2) An extra tax should be levied on petrol from the road users to develop a road development fund called **'Central Road Fund'**
3) A Semi-official technical body should be formed to pool technical know-how from various parts of the country and to act as an advisory body on various aspects of roads.
4) A research organization should be instituted to carry out research and development work pertaining to roads and to be available for consultations.

IMPLEMENTATIONS:

Majority of the recommendations were accepted by the government implemented by Jayakar Committee.

Some of the technical bodies were formed such as,

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

CENTRAL RESEARCH FUND (CRF):

1) Central Research Fund (CRF) was formed on 1st March 1929
2) The consumers of petrol were charged an extra levy of 2.64 paisa/liter of petrol to build up this road development fund.
3) 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.
4) 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

5) The accounts of the CRF are maintained by the Accountant General of Central Revenues.
6) The control of the expenditure is exercised by the Roads Wings of Ministry of Transport.

INDIAN ROAD CONGRESS (IRC):

1) It is a semi-official technical body formed in 1934. It was formed to recommend standard specifications.
2) 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.
3) IRC has played an important role in the formulation of the 20-year road development plans in India.
4) Now, it has become an active body of national importance controlling specifications, guidelines and other special publications on various aspect of Highway Engineering.

CENTRAL ROAD RESEARCH INSTITUTE (CRRI):

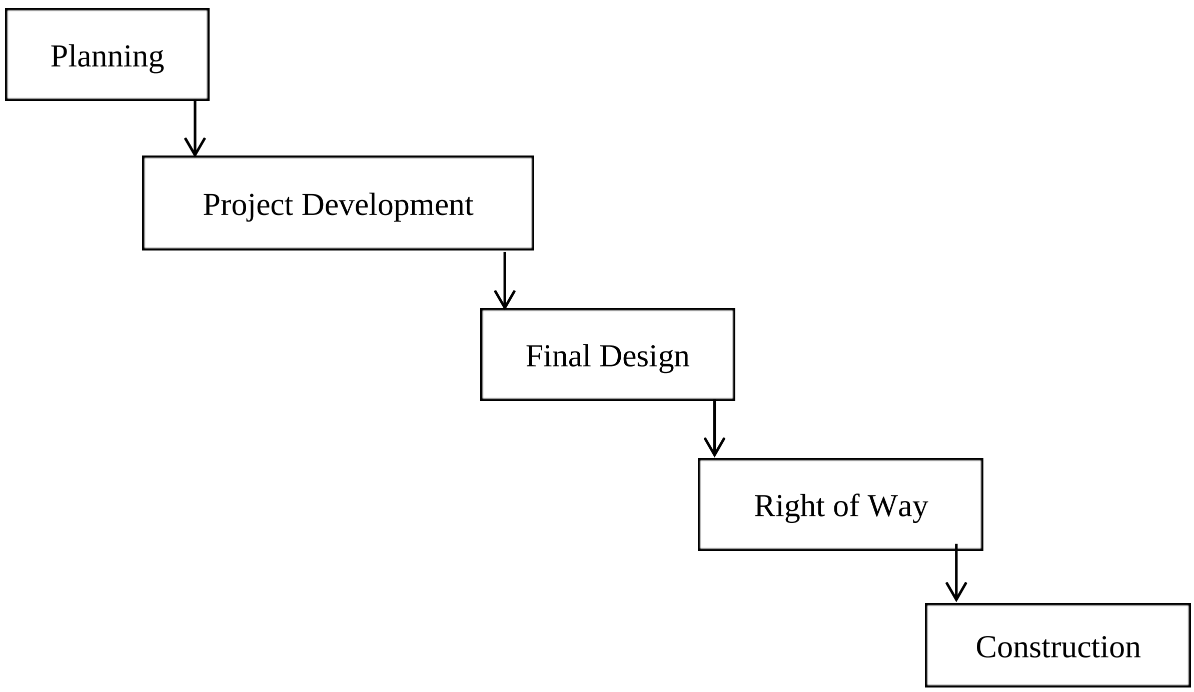
1) CRRI was formed in the year 1950 at New Delhi
2) It was formed for research in various aspect of highway engineering
3) It is one of the National laboratories of the Council of Scientific and Industrial Research.
4) This institute is mainly engaged in applied research and offers technical advice to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

HIGHWAY DEVELOPMENT AND PLANNING

INTRODUCTION

Highway design is only one element in the overall highway development process. Historically, detailed design occurs in the middle of the process, linking the preceding phases of planning and project development with the subsequent phases of right-of-way acquisition, construction, and maintenance. While these are distinct activities, there is considerable overlap in terms of coordination among the various disciplines that work together, including designers, throughout the process.

It is during the first three stages, planning, project development, and design, that designers and communities, working together, can have the greatest impact on the final design features of the project. In fact, the flexibility available for highway design during the detailed design phase is liBRCEd a great deal by the decisions made at the earlier stages of planning and project development. This Guide begins with a description of the overall highway planning and development process to illustrate when these decisions are made and how they affect the ultimate design of a facility.



Objectives of Highway Planning

Planning if considered as pre-requisite before attempting any development program in the present era. Highway planning is of great importance when funds available are liBRCEd whereas the total planning is of great importance when the funds are liBRCEd whereas the total requirement is much higher. The objectives are as follows

a) To plan the overall road network for efficient and safe traffic operations, but at minimum cost. Here the costs of construction, maintenance and resurfacing or strengthening of pavement layers and vehicle operation costs are taken into consideration.
b) To arrive at the road system and the lengths of different categories of roads which could provide maximum utility and could be constructed within the available resources during the plan period under construction
c) To divide the overall plan into phases and to decide priorities.
d) To fix up date wise priorities for development of each road link based on utility as the main criterion for phasing the road development program.
e) To plan for the future requirements and improvements of roads in view of anticipated developments.
f) To work out suitable financing systems

Phases of Highway Planning

Highway planning includes the following phases

- Assessment of road length requirement for an area.
- Preparation of masterplan showing the phasing of plan in five year plans or annual plans.

MEANING OF HIGHWAY AND ROAD

Road: A **road** is a thoroughfare, route or way on land between two places, which typically has been paved or otherwise improved to allow travel by some conveyance, including a horse, cart, or motor vehicle.

Highway: A **highway** is a public road, especially a major road connecting two or more destinations. Any interconnected set of highways can be variously referred to as a "highway system", a "highway network", or a "highway transportation system". Each country has its own national highway system.

CLASSIFICATION OF ROADS

Types of Roads

Basically, different types of roads can be classified into two categories namely,

a) All-weather roads and
b) Fair-weather roads.

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.

a) Based on the Carriage Way

- **Paved Roads**: These are the roads which have a hard pavement surface on the carriage way
- **Unpaved Roads:** These are the roads without the hard pavement surface on the carriage way, usually they are earthen or gravel roads.

b) Based on Surface Pavement Provided,

- **Surface Roads:** These roads are provided with any type of bituminous or cement concrete surfacing.
- **Unsurfaced Roads:** These roads are not provided with a bituminous orcement concrete surfacing.

Roads which are provided with bituminous surfacing are called as **Black Toped Roads** and that of concrete are referred to as **Concrete Roads** respectively

Methods of Classification of Roads

The roads are generally classified based on the following

a) Traffic Volume

b) Load transported of tonnage

c) Location and function

a) Based on Traffic Volume: The classification based on traffic volume or tonnage have been arbitrarily fixed by different agencies and are classified as

- Heavy
- Medium
- Light traffic roads

b) Based on Load transported or tonnage:

- Class-I or Class-A
- Class-II or Class-B.

c) Based on location and Function:

The Nagpur Road Plan classified the roads in India into the following categories

1) **National Highways (NH):** The NH connects the capital cities of the states and the capital cities to the port. The roads connecting the neighboring countries are also called as NH. The NH are at least 2 lanes of traffic about 7.5m d wide. The NH are having concrete or bituminous surfacing.

2) **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.

3) **Major District Roads (MDR):** These roads connect the areas of production and markets with either a SH or railway. The MDR should have at least metaled single lane carriage way (i.e., 3.8m) wide. The roads carry mixed traffic.

4) **Other District Roads (ODR):** these roads connect the village to other village or the nearest district road, with ghat, river etc. these roads have a single lane and carry mixed traffic.

5) **Village Roads (VR):** these roads, like other district roads, connect the village or village or nearby district road. The roads carry mixed traffic.

Modified Classification of Road System by Third Road Development Plan

The road classification system was modified in the third 20-year development plan. The roads are now classified into three classes and are as follows

1. Primary System
   - Expressways
   - National Highways (NH)
2. Secondary System
   - State Highways (SH)
   - Major District Roads (MDR)
3. Tertiary System
   - Other District Roads (ODR)
   - Village Roads

Classification of Urban Roads

The road system within urban areas are classified as Urban Roads and will form a separate category of roads taken care by respective urban authorities. The lengths of urban roads are not included in the targets of the 3rd 20-year road development plan 1981-2001.

a) Arterial roads
b) Sub-arterial roads
c) Collector Streets
d) Local Streets

- Arterial and Sub-arterial roads are primarily for through traffic on a continuous route, but sub-arterials have a lower level of traffic mobility than the arterials.
- Collector streets provide access to arterial streets and they collect and distribute traffic from and to local streets which provide access to abutting property.

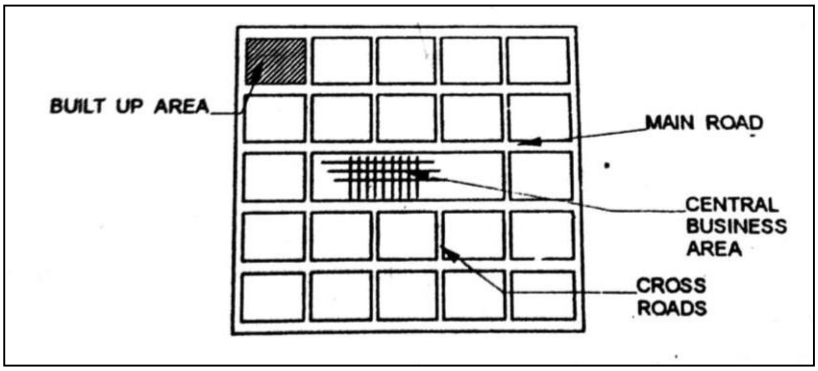
ROAD PATTERNS

There are various types of road patterns and each pattern has its own advantages and limitations. The choice of the road pattern depends upon the various factors such as:

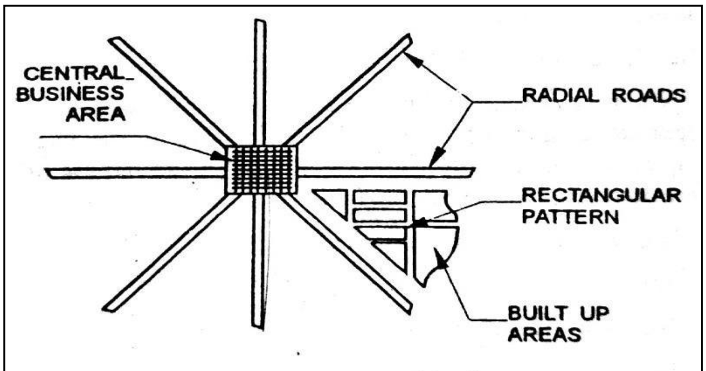
- Locality
- Layout of the different towns, villages, industrial and production centers.
- Planning Engineer.

The various road patterns may be classified as follows:

1) **Rectangular or block pattern:** In this, entire area is divided into rectangular segments having a common central business and marketing area. This area has all the services located in the central place. This pattern is not convenient or safe from traffic operation point of view and it results into more number of accidents at intersections. E.g.: Chandigarh city.



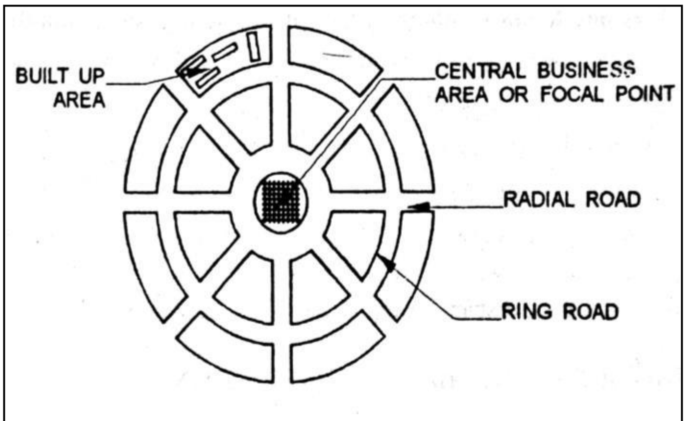
2) **Radial or star and block pattern**: In this, roads radially emerge from the central business area in all directions and between two built-up area will be there. The main advantage in this, central place is easy accessible from all the directions. E.g.: Nagpur



(c) Radial or star and block pattern

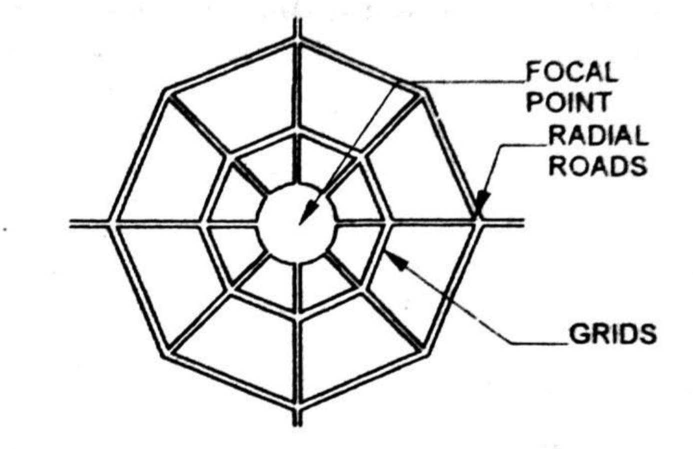
3) **Radial or star and circular pattern:** In this road radiate in all the directions and also circular ring roads are provided.

Advantages: Traffic will not touch the heart of the city and it flows radially and reaches the other radial road and thereby reducing the congestion in the center of the city. This ring road system is well suited for big cities where traffic problems are more in the heart of the city. E.g.: Connaught place in New Delhi.



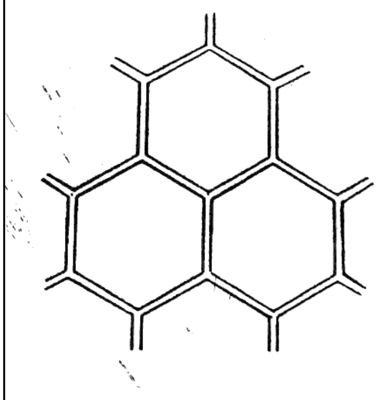
(d) Radial or star and circular pattern

4) **Radial or star and grid pattern**: It is very much similar to star and the circular pattern expects the radial roads are connected by grids. In this pattern, a grid is formed around the central point which is a business center. E.g. Nagpur road plan.



(e) Radial or star and grid pattern

5) **Hexagonal pattern**: In this entire zone of planning is divided into hexagonal zones having separate marketing zone and central services surrounded by hexagonal pattern of roads. Each hexagonal element is independent. At each corner of hexagon three roads meet.



(b) Hexagonal pattern

6) **Minimum travel pattern**: In this type, city is divided into number of nodal points around a central portion by forming sectors. And each sector is divided again in such a way that from each of the nodal center, the distance to the central place is minimum.

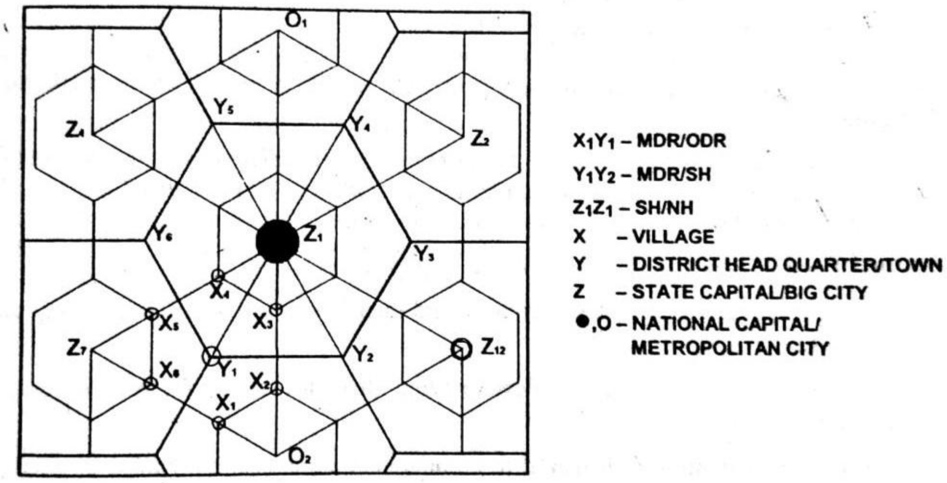


Fig. 2.3 Concept of star and grid pattern

PLANNING SURVEYS

The studies for collecting the factual data for highway planning are known as **'Fact Finding Studies' or 'Planning Surveys'**. The fact-finding studies point to an intelligent approach for planning and these studies should be carried if the highway programme is to be protected from inconsistent and short-sighted policies. Planning based on the factual data and analysis may be considered scientific and sound.

Objectives of Planning Surveys:

- Workout, the financial system and recommended changes in tax arrangements and budget procedures, provide efficient, safe economics, comfortable and speedy movement for goods and people.
- Plan a road network for efficient traffic operation at minimum cost.
- Plan for future requirements and improvements of roads in view of developments and social needs.
- Fix up data wise priorities for development of each road link based on their utilities.

The planning surveys consist of the following studies:

a) Economic Studies

The details to be collected during the economic studies are useful in estimation of the requirements, cost involved for the proposed highway improvement programme and economic justification.

This study consists the following details:

a) Population and its distribution
b) Trend of population growth
c) Age and land products
d) Existing facilities
e) Per Capita income.

b) Financial Studies

The financial studies are essential to study the various financial aspects such as sources of income, various types of revenues from duties and taxes on products, road transport, vehicle registration, court fees etc. and the future trends. This study involves collecting the details such as:

a) Sources of income
b) Living Standards
c) Resources from local levels
d) Factor trends in financial.

c) Traffic or Road Use Studies

All the details of the existing traffic, such as classified traffic volume, growth rate of different vehicle classes, pattern of flow or origin destination characteristics, particulars of passenger trips and goods movements, existing facilities for mass transportation, trend in road accidents, accidents costs etc. The detail collected are as follows

a) Classified traffic volume in vehicles per day, annual average daily traffic, peak and design hourly volume
b) Origin and destination studies based on home interview method
c) Traffic flow pattern
d) Mass transportation facilities
e) Accidents, their causes and cost analysis
f) Future trend and growth in traffic volume and goods traffic, trend in traffic pattern
g) Growth of passenger trips and the trend in the choice of modes

d) Engineering Studies

All the details of the topography, soil and drainage characteristics, alignment of the existing roads, deficiencies in drainage, alignments and geometrics of existing roads and requirements of essential upgradation, identification of maintenance and problems etc.,

This involves:

a) Topographic study and Soil details
b) Location and classification of existing roads

c) Assessment of various other developments in the area that are likely due to the proposed highway development
d) Road life studies
e) Specific problems in drainage constructions and maintenance.

PREPARATION OF PLANS

The details collected during the planning surveys are tabulated and plotted on the maps of the area under planning. Before finalizing the alignment and other details of the road development program, the information collected during the fact-finding studies are presented in the form of various plans. They are as follows

Plan-1: General area plan showing most of the existing details about the topographical details related to existing road network, drainage, structures, towns and villages with population, agricultural, industrial and commercial activities.

Plan-2: Plan showing the distribution of population groups in accordance with the categories made in appropriate plan.

Plan-3: Plan showing the locations of places with their respective quantities of productivity.

Plan-4: Should indicate the existing network of roads and proposals received.

Ultimately, the Master plan is the one to be implemented.

MASTER PLAN

Master plan is referred to as road development plan of a city; district or a street or for whole country. It is an ideal plan showing full development of the area at some future date. It serves as the guide for the plan to improve some of the existing roads and to plan the network of new roads.

It helps in controlling the industrial, commercial and agricultural and habitat growth in a systematic way of that area. It gives a perceptive picture of a fully developed area in a plan and scientific way.

Stages in the preparation of master plan:

- Data Collection: It includes data regarding existing land use, industrial and agricultural growth, population, traffic flow, topography, future trends.
- Preparation of draft plan and invite suggestions and comments from public
- Revision of draft plan in view of the discussions and comments from experts and public.
- Comparison of various alternate proposals of road system and finding out the sequence in which the master plan will be implemented.

  In India, targeted road lengths were fixed in various road plans, based on population, area and agricultural and industrial products. The same way it may be taken as a guide to decide the total length of road system in each alternate proposal while preparing a master plan for a town or locality.

SATURATION SYSTEM

In this system optimum road length is calculated for an area based on the concept of attaining maximum utility per unit length of the road. This is also called as **MAXIMUM UTILITY SYSTEM.**

Factors to attain maximum utility per unit length are:

a) Population served by the road network
b) Productivity served by the network
- Agricultural Products
- Industrial Products

The various steps to be taken to obtain maximum utility per unit length are:

1) **Population factors or units:** Since, the area under consideration consists of villages and towns with different population these are grouped into some convenient population range and some reasoning values of utility units to each range of population serve are assigned.

a) Population less than 500, utility unit = 0.25
b) 501 to 1001, utility unit = 0.50
c) 1001 to 2000, utility unit = 1.00
d) 2001 to 5000, utility unit = 2.00 etc.

2) **Productivity Factors or units:** The total agricultural and industrial products served by each road system are worked out and the productivity served may be assigned appropriate values of utility units per unit weight.

3) **Optimum Road length:** Based on the master plan the targeted road length is fixed for the country on the basis of area or population and production or both. And the same may be taken as a guide to decide the total length of the road system in each proposal.

Problems in Class Notes

PHASING ROAD DEVELOPMENT IN INDIA

The first attempt for proper planning of the highway development programme in India on a long-term basis was made at the Nagpur Conference in 1943. After the completion of the Nagpur Road Plan targets, the Second Twenty year Plan was drawn for the period 1961- 1981. The Third Twenty Year Road Development Plan for the period 1981-2001 was approved only by the year 1984.

The fourth 20-year road development plan of the country for the period 2001 – 2021 has not yet been approved as an official plan document, instead ‘Roads Development Vision: 2021’ has been formulated.

First 20-Year Road Plan (Nagpur Road plan)

This plan was formed in the year 1943 at Nagpur and plan period was from 1943- 1963.Two plan formulae were finalized at the Nagpur Conference for deciding two categories of road length for the country as a whole as well as for individual areas (like district). This was the first attempt for highway planning in India. The two plan formulae assumed the Star and Grid pattern of road network. Hence, the two formulae are also called “Star and Grid Formulae”.

Salient Features of Nagpur Road Plan

All the roads were classified into 5 categories namely

1) National Highways (NH)
2) State Highways (SH)
3) Major District Roads (MDR)
4) Other District Roads (ODR)
5) Village Roads (VR)

Two plan formulas were suggested for deciding the length of two categories of roads as given below

Category – 1: Surfaced or metaled roads meant for NH/SH/MDR

Category – 2: Unsurfaced roads meant for ODR/VR

Nagpur road plan aimed at achieving a modest average road density of 16km per km^2 area.

Second Twenty Year Road Plan (Bombay Road Plan):

As the target road length of Nagpur road plan was completed nearly earlier in 1961 a long-term plan was initiated for twenty-year period which was initiated by IRC. Hence, the second twenty year road plan came into picture which was drawn for the period of 1961-81. The second twenty year road plan was envisaged overall road length of 10, 57,330 km by the year 1981.

Salient Features of Second 20-year Road Plan:

- Every town with population above 2000 in plains should be connected by a bituminous road or metaled road, above 1000 in semi-hilly area above 500 in hilly area
- 1600 km length of expressways was proposed.
- Development allowance is 5% only
- Length of railway track was not deducted.
- Five equations are given to find NH/SH/MDR/ODR/VR.

Third Twenty Year Road Plan (Lucknow Road Plan):

The Third twenty year road plan was prepared by the Road Wing of the Ministry of Shipping and Transport with the active co-operation from a number of organizations and the experts in the field of Highway Engineering and Transportation. This document was released during the 45th Annual Session and the Golden Jubilee celebrations of the Indian Road Congress in February 1985 at Lucknow. Therefore, this plan for 1981-2001 is also called as **'Lucknow Road Plan'.**

Salient Features of Second 20-year Road Plan

- The future road development should be based on the revised classification of road system consisting of Primary, secondary and tertiary systems.
- The road network should be developed so as to preserve the rural oriented economy and to develop small to towns with all the facilities. All the villages with population above 500 should be connected with all-weather roads by the end of the century.
- The overall road density should be increased to 82 km per 100 sq. km area by the year 2001 and 40km for hill areas of altitude up to 2100m and 15km for altitude over 2100m.
- The NH network should be expanded to form square grids of 100km sides so that no part of the country in more than 50km away from a NH
- The length of SH and MDR required should be decided based on the areas and no. of towns with population above 5000 in the state or region.
- Expressway should be constructed along the major traffic corridors to provide fast travel.
- All the towns and villages with population above 1500 should be connected by MDR and villages with population 1000 to 1500 by ODR. There should be road within a distance of 3km in plain and 5km in hilly terrain connecting the villages with population less than 500.

- Roads should be built in less industrialized areas to attract the growth of industries.
- Long term master plans for road development should be prepared at various level i.e., taluk, district, state and national level. The road network should be scientifically decided to provide maximum utility.
- The existing roads should be improved by rectifying the defects in the road geometrics, widening of the pavements, improving the riding quality of the pavement surface and strengthening of pavement structure
- There should be improvements in environmental quality and road safety.

Road length by 3rd 20-year road development plan

a) Length of NH – 1km per 50sq. km area.

b) Length of SH

1) By total area – SH, km = Area of the state, sq.km/25

2) By total no of town and area in the state, SH, km =

$$\frac{(62.5 \text{ x No towns in the state} - \text{ area of the state, sq. km})}{50}$$

Adopt length of SH (higher of the two criteria)

c) Length of MDR

1) By total area – MDR, km = Area of the state, sq.km/12.5

2) By total no of town and area in the state, MDR, km = 90 x No. of towns in the state.

Adopt length of SH (higher of the two criteria).

PRESENT SCENARIO OF ROAD DEVELOPMENT IN INDIA

NATIONAL HIGHWAY DEVELOPMENT PROJECTS (NHDP)

- Realizing the deficiencies in the National Highway System in the country the National Highways Authorities of India (NHAI) took up the National Highways Development Projects (NHDP) by the year 2000 in different phases
- The **National Highways Development Project** (NHDP) is a project to upgrade, rehabilitate and widen major highways in India to a higher standard.
- The project was started in 1998 under the leadership of then Prime Minister, Atal Bihari

Vajpayee. National Highways account for only about 2% of the total length of roads, but carry about 40% of the total traffic across the length and breadth of the country.

- This project is managed by the National Highways Authority of India (NHAI) under the Ministry of Road, Transport and Highways.
- The NHDP represents 49,260 km of roads and highways work and construction in order to boost economic development of the country.
- The government has planned to end the NHDP program in early 2018 and consume the ongoing projects under a larger Bharatmala project.

Phase I: Golden Quadrilateral of total length 5846km connecting the 4 major metropolitan cities. The four sides of the quadrilateral are Delhi – Mumbai, Mumbai – Chennai (Via Bengaluru), Chennai – Kolkata and Kolkata- Delhi.

Phase II: North-South and East-West corridors comprising national highways connecting four extreme points of the country. The North–South and East–West Corridor (NS-EW; 7,142 km) connecting Srinagar in the north to Kanyakumari in the south, and Silchar in the east to Porbandar in the west. Total length of the network is 7,142 km.

Phase III: The government on 12th April, 2007 approved NHDP-III to upgrade 12,109 km (7,524 mi) of national highways on a Build, Operate and Transfer (BOT) basis, which takes into account high-density traffic, connectivity of state capitals via NHDP Phase I and II, and connectivity to centers of economic importance.

Phase IV: The government on 18th June, 2008 approved widening 20,000 km of highway that were not part of Phase I, II, or III. Phase IV will convert existing single-lane highways into two lanes with paved shoulders.

Phase V: As road traffic increases over time, a number of four-lane highways will need to be upgraded/expanded to six lanes. On 5 October, 2006, the government approved for upgrade of about 5,000 km (3,100 mi) of four-lane roads.

Phase VI: The government is working on constructing 1,000 km (620 mi) expressways that would connect major commercial and industrial townships. It has already identified 400 km (250 mi) of Vadodara (earlier Baroda)-Mumbai section that would connect to the existing Vadodara (earlier Baroda)-Ahmedabad section. The World Bank is studying this project. The project will be funded on BOT basis. The 334 km (208 mi) Expressway between Chennai—Bangalore and 277 km (172 mi) Expressway between Kolkata—Dhanbadhas been identified and feasibility study and DPR contract has been awarded by NHAI.

Phase VII: This phase calls for improvements to city road networks by adding ring roads to enable easier connectivity with national highways to important cities. In addition, improvements will be made to stretches of national highways that require additional flyovers and bypasses given population and housing growth along the highways and increasing traffic. The government has planned to invest Rs. 16,680 Cr for this phase. The 19 km (12 mi) long Chennai Port—Maduravoyal Elevated Expressway is being executed under this phase.

PRADHAN MANTRI GRAM SADAK YOJANA (PMGSY)

- An accelerated village road village road development called Pradhan Mantri Gram Sadak Yojana was launched by the Govt. of India in Dec 2000 under the guidance of Ex. Prime Minister Shri Atal Bihari Vajapayee to provide villages with all-weather roads.
- The ministry of Rural Development was given the responsibility to prepare the master plans in consultation with the State Governments.
- The objective of PMGSY was to provide connectivity to all unconnected habitations having a population of 500 and above with all-weather roads.
- The above population limit is relaxed in the case of hills, tribal and desert areas of the country.
- The Pradhan Mantri Gram Sadak Yojana (PMGSY) is a 100% Centrally Sponsored Scheme. 0.75□ / litre out of the Cess on High Speed Diesel (HSD) is earmarked for this Programme.

PROGRAMME OBJECTIVES

1) The primary objective of the PMGSY is to provide Connectivity, by way of an All-weather Road (with necessary culverts and cross-drainage structures, which is operable throughout the year), to the eligible unconnected Habitations in the rural areas with a population of 500 persons and above in Plain areas.
2) In respect of the Hill States (North-East, Sikkim, Himachal Pradesh, Jammu & Kashmir and Uttarakhand), the Desert Areas (as identified in the Desert Development Programme), the Tribal (Schedule V) areas and Selected Tribal and Backward Districts (as identified by the Ministry of Home Affairs and Planning Commission).
3) The objective would be to connect eligible unconnected Habitations with a population of 250 persons and above.

KSHIP

- The **Karnataka State Highways Improvement Project (KSHIP)** is an initiative of the Public Works Department of the Government of Karnataka for improvement of road network of the state with World Bank assistance.
- The Public Works Department carried out Strategic Option Study (SOS) during 1996 on a road network of 13,362 km comprising State Highways and Major District Roads and the study identified 2888 km of roads for prioritized improvements.
- The World Bank have extended Technical Assistance (T.A.) Loan of US $ 3.2 million for project preparation through the Department of Economic Affairs of Ministry of Finance, Government of India for taking up the Project Coordinating Consultancy (PCC) Services to investigate and prepare detailed project report on the 2888 km and Institutional Development Strategy (IDS) Study.
- With concurrence of the World Bank, M/s. Scott Wilson Kirkpatrick, United Kingdom were selected and appointed on 07-01-1999 to carryout PCC Services for 2505 km of roads.
- The PCC Services were divided into Phase I & II. The PCC Consultants carried out feasibility and social & environmental screening and identified 2271 km for prioritized improvement. It is proposed to undertake Upgradation of 992 Km and Rehabilitation of 1277 Km.

KRDCL

Karnataka Road Development Corporation (KRDCL) was incorporated on 21st of July 1999 as a wholly owned Government of Karnataka Company as per the Provisions of the Company's Act, 1956

- KRDCL is a company under the Public Works, Ports & Inland Water Transport Department. This Company was established to promote surface infrastructure by taking up Road Works, Bridges etc., and to improve road network by taking up construction widening and strengthening of roads, construction of bridges, maintenance of roads etc., and to take up projects on BOT, BOOT, BOLT.
- Since inception Karnataka Road Development Corporation LiBRCEd has strived to improve the road network and to establish connectivity to all the nook & corner of the State.

ROAD DEVELOPMENT PLAN: VISION 2021

- Actual achievement in terms of length of different categories of roads in the country at the end of the 3rd 20-year road development plan period was compared with the plan targets.
- It was observed that actual length of NH and SH achieved fell short of plan targets. The total length of NH achieved was 57,700km as against the target of 66,000km and that the SH achieved was 1,24,300km as against of 1,45,000km.
- This vision document has considered the need for overall development of road system in country. The total target length of primary and secondary road system to be achieved in the country by the year 2020 are given below
- Primary highway system consisting of 15,766km of expressway and 80,000km of NH
- Secondary road system consisting of 1,60,000 km of SH and 3,20,000km of MDR
- The above document also has given special attention for road development needs in North-Eastern regions and other isolated areas. In view of rapid growth rate of urban centres, some suggestion has been made for the development of urban road system also.
- Tertiary system of rural roads consisting of ODR and VR are to be developed in order to provide all-weather road connectivity to all the villages of the country in a phased manner. Considering the importance of this subject, a rural road development plan document was prepared.

Rural Road Development Plan: Vision 2025

It was developed for the 20-year period of 2005-2025 to provide basic access to villages in phases:

- Phase – I: Villages with population above 1000
- Phase – II: Villages with population above 500
- Phase – III: Villages with population below 500

Lower population limits were fixed for under developed regions including hills, deserts and tribal areas.

MODULE – 2

HIGHWAY ALIGNMENT AND SURVEYS

Introduction

The position or the layout of the central line of the highway on the ground is called the alignment. Highway Alignment includes both

a) Horizontal alignment includes straight and curved paths, the deviations and horizontal curves.
b) Vertical alignment includes changes in level, gradients and vertical curves.

A new road should be aligned very carefully as improper alignment will lead to increase in construction, maintenance and vehicle operation cost. Once the road is aligned and constructed, it is not easy to change it due to increase in cost of adjoining land and construction of costly structures by the roadside

Requirements

The requirements of an ideal alignment are

a) Short: The alignment between two terminal stations should be short and as far as possible be straight, but due to some practical considerations deviations may be needed.

b) Easy: The alignment should be easy to construct and maintain. It should be easy for the operation of vehicles. So, to the maximum extend easy gradients and curves should be provided.

c) Safe: It should be safe both from the construction and operating point of view especially at slopes, embankments, and cutting. It should be safe for traffic operation with safe geometric features.

d) Economical: The alignment should be economical and it can be considered so only if the total life cycle cost considering the initial cost, maintenance cost, and vehicle operating cost is lowest.

Factors Controlling Alignment

For an alignment to be shortest, it should be straight between the two terminal stations, but this is not always possible due to various practical difficulties such as intermediate obstructions or topography. A road which is economical with low initial investment may not be the most economical in terms of maintenance or vehicle operation cost(VOC). Thus, is may be seen that an alignment can fulfil all the requirements simultaneously, hence a judicial choice is made considering all the factors.

The various factors that control the alignment are as follows:

a) Obligatory Points
b) Traffic
c) Geometric Design
d) Economics
e) Other Considerations

Obligatory Points

These are the control points governing the highway alignment. These points are classified into two categories.

1) Points Through Which the Alignment Should Pass
2) Points Through Which the Alignment Should Not Pass.

Points Through Which the Alignment Should Pass

a) Bridge site: The bridge can be located only where the river has straight and permanent path and also where the abutment and pier can be strongly founded. The road approach to the bridge should not be curved and skew crossing should be avoided as possible. Thus, to locate a bridge the highway alignment may be changed.
b) Mountain: While the alignment passes through a mountain, the various alternatives are to either
c) Construct a tunnel or to go around the hills. The suitability of the alternative depends on factors like topography, site conditions and construction and operation cost.
d) Intermediate town: The alignment may be slightly deviated to connect an intermediate town or village nearby. These were some of the obligatory points through which the alignment should pass.

Points Through Which the Alignment Should Not Pass.

a) Religious places: These have been protected by the law from being acquired for any purpose. Therefore, these points should be avoided while aligning.
b) Very costly structures: Acquiring such structures means heavy compensation which would result in an increase in initial cost. So, the alignment may be deviated not to pass through that point.
c) Lakes/ponds etc.: The presence of a lake or pond on the alignment path would also necessitate deviation of the alignment.

Traffic

The alignment should suit the traffic requirements. Based on the origin- destination data of the area, the desire lines should be drawn. The new alignment should be drawn keeping in view the desire lines, traffic flow pattern etc.

Geometric design

Geometric design factors such as gradient, radius of curve, sight distance etc. also governs the alignment of the highway. To keep the radius of curve minimum, it may be required to change the alignment of the highway. The alignments should be finalized such that the obstructions to visibility do not restrict the minimum requirements of sight distance. The design standards vary with the class of road and the terrain and accordingly the highway should be aligned.

Economics

The alignment finalized should be economical. All the three costs i.e. construction, maintenance, and operating cost should be minimum. The construction cost can be decreased much if it is possible to maintain a balance between cutting and filling. Also try to avoid very high embankments and very deep cuttings as the construction cost will be very higher in these cases.

Other Considerations

The various other factors that govern the alignment are drainage considerations, political considerations and monotony. The vertical alignment is often guided by drainage considerations such as sub surface drainage, water level, seepage flow, and high flood levels. A foreign territory coming across the alignment will necessitate the deviation of the horizontal alignment. In flat terrain, even though it is possible to have a very long stretch of road which is absolutely straight may be monotonous for driving. Hence it is recommended to have a slight bend or road side amenities to break monotony.

ENGINEERING SURVEYS FOR HIGHWAY ALIGNMENT

Stages of Engineering Surveys

Before a highway alignment is finalised in a new highway project, engineering surveys are to be carried out. These engineering surveys may be completed in the following four stages:

a) Map Study
b) Reconnaissance Survey
c) Preliminary Surveys
d) Final Location and Detailed Surveys

Map Study

It is possible to suggest the likely routes of the roads if the topographic map of the area is available. In India, topographic maps are available from the Survey of India, with 15 or 30 metre contour intervals. The main features like rivers, hills valleys, etc. are also shown on these maps.

The probable alignment can be located on the map from the following details available on the map.

- Alignment avoiding valleys, ponds or lakes
- When the road has to cross a row of hills or mountains, possibility of crossing through a mountain pass.
- Approximate location of bridge site for crossing rivers, avoiding bend of the river, if any
- When a road is to be connected between two stations, one of the top and the other on the foot of the hill, then alternate routes can be suggested keeping in view the design or ruling gradient and the maximum permissible gradient

Thus, from the map study alternate routes can be suggested. It may also be possibly from map study to drop a certain route in view of any unavoidable obstructions or undesirable ground and map study gives a rough guidance of the routes to be further surveyed in the field.

Reconnaissance Survey

The second stage of engineering surveys for highway alignment is the reconnaissance survey. During the reconnaissance, the engineer visits the site and examines the general characteristics of the area before 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, very simple survey instruments are used by the reconnaissance party to collect additional details rapidly, but not accurately. All relevant details which are not available in the map are collected and noted down. Some of the details to be collected during reconnaissance are given below

a) Valleys, ponds, lakes, marshy land, ridge, hills, permanent structures and other obstructions alone the route which are not available in the map
b) Approximate values of gradient, length of gradients and radius of curves of alternate alignments.
c) Number and type of cross drainage structures, maximum flood level and natural ground water level along the probable routes.
d) Soil type along the routes from field identification tests and observation of geological features
e) Sources of construction materials, water and location of stone quarries
f) When the road passes through hilly or mountainous terrain, additional data regarding the geological formation, type of rocks, dip of strata, seepage flow etc. may be observed so as to decide the stable and unstable sides of the hill for highway alignment

A rapid reconnaissance of the area, especially when it is vast and the terrain is difficult and it may be done by aerial survey. From the details collected during the survey the alignment proposed may be altered or even changes completed.

Preliminary Survey

The main objectives of the preliminary survey are

- To survey the various alternate alignment 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 earthwork materials and other construction aspects and to work out the cost of alternate proposals.

The preliminary survey may be carried out by of following methods

a) Conventional approach, in which a survey party carries out surveys using the required field equipment, taking measurements, collecting topographical and other data and carrying out soil survey
b) Rapid approach, by aerial survey taking the required aerial photographs and by photogrammetric methods and photo-interpretation techniques for obtaining the necessary topographic and other maps including details of soil and geology
c) Modem techniques by use of Global Positioning System (GPS)

The procedure of the conventional methods of preliminary survey is given in following steps:

a) Primary Traverse
b) Topographical Features
c) Levelling Work
d) Drainage Studies and Hydrological Data
e) Soil Survey
f) Material Survey
g) Traffic Studies Primary Traverse

Primary Traverse

The first step in the preliminary survey is to establish the primary traverse, following the alignment recommended in the reconnaissance. For alternate alignments either secondary traverses or independent primary traverses may be necessary. As these traverses are open traverses and adjustment of errors is not possible later, the angles should be very accurately measured using a precision theodolite.

Topographic Features

After establishing the centre lines of preliminary survey, the topographical features are recorded. All geographical and other man-made features along the traverse and for a certain width on either side are surveyed and plotted. The width to be surveyed is generally decided by the survey party, but the absolute minimum width is the land width of the proposed alignment.

Levelling work

Levelling work is also carried out side by side to give the centre line profiles and typical cross sections. Permanent and temporary bench marks should be first established at appropriate locations and the levels should be connected to the GTS datum. The levelling work in the preliminary survey is kept to a minimum just sufficient to obtain the approximate earth work in the alternate alignments. To draw contours of the strip of land to be surveyed, cross section levels should be taken at suitable intervals, generally 100 to 200 m in plain terrain, up to 50 m in rolling terrain and up to 30 m in hilly terrain.

Drainage Studies and Hydrological Data

Drainage investigations and hydrological data are collected so as to estimate the type, number and approximate size of cross drainage structures. Also, the vertical alignment of the highway, particularly the grade line is decided based on the hydrological and drainage data, such as HFL. ponded water level, depth of water table, amount of surface runoff, etc.

Soil Survey

Soil survey is an essential part of the preliminary survey as the suitability of the proposed location is to be finally decided based on the soil survey data. The soil survey conducted at this stage also helps in working out details of earth work, slopes, suitability of materials, subsoil and surface drainage requirements and pavement type and the approximate thickness requirements. All these details are required to make a comparative study of alternate proposals. A detailed soil survey is not necessary. Post-hole auger or any other suitable types of hand augers may be used depending on the soil type to collect the soil sample up to a depth of 1 to 3 metre below the likely finished road level or the existing ground level, whichever is lower. When the road is expected to be constructed over an embankment, the depth of exploration should extend up to twice the height of embankment from the ground level. During the soil exploration if the ground water table is struck, the depth from the ground surface is also noted. The types of soils encountered along the route up to the depth under consideration are marked on the soil profile either symbolically or by suitable colour coding.

Material Survey

The survey for naturally occurring materials like stone aggregates, soft aggregates, etc. and identification of suitable quarries should be made. Also, availability of manufactured materials like cement, lime, brick, etc. and their locations may be ascertained.

Traffic Survey

Traffic surveys conducted in the region form the basis for deciding the number of traffic lanes and roadway width, pavement design and economic analysis of the highway project. Traffic volume counts of the classified vehicles are to be carried out on all the existing roads in the region, preferably for 24 hours per day for seven days. Origin and destination surveys are very useful for deciding the alignment of the roads. This study may be earned out on a suitable sample of vehicle users or drivers. In addition, the required traffic data may also be collected so that the traffic forecast could be made for 10 to 20 year periods.

Determination of Final Centre Line

After completing the preliminary surveys and conducting the comparative studies of alternative alignments, the final centre line of the road is to be decided in the office before the final location survey. For this, the preliminary survey maps consisting of contour plans, longitudinal profile and cross sections of the alternate alignments should be prepared and carefully studied to decide the best alignment satisfying engineering aesthetic and economical requirements. After selecting the final alignment, the grade lines are drawn and the geometric elements of the horizontal and vertical alignments of the road are designed.

Rapid method using aerial survey and modern technique using GPS

Aerial photographic surveys and photogrammetric methods are very much suited for preliminary surveys, especially when the distance and area to be covered are vast, The survey may be divided into the following steps:

Taking aerial photographs of the strips of land to be surveyed with the required longitudinal and lateral overlaps. Vertical photographs are necessary for the preparation of mosaics.

a) The photographs are examined under stereoscopes and control points are selected for establishing the traverses of the alternate proposals. The control points are located on the maps
b) Using stereo-pair observations, the spot levels and subsequently contour details may be noted down on the maps
c) Photo-interpretation methods are used to assess the geological features, soil conditions, drainage requirements etc.

Final Location and Detailed Survey

The alignment finalised at the design office after the preliminary survey is to be first located on the field by establishing the centre line. Next detailed survey should be carried out for collecting the information necessary for the preparation of plans and construction details for the highway project.

Location

The centre line of the road finalised in the drawings is to be transferred on the ground during the location survey. This is done using a transit theodolite and by staking of the centre line. The location of the centre line should follow, as closely as practicable, the alignment finalised after the preliminary surveys. Major and minor control points are established on the ground and centre pegs are driven, checking the geometric design requirements. However, modifications in the final location may be made in the field, if found essential. The centre line stakes are driven at suitable intervals, say at 50 metre intervals in plain and rolling terrains and at 20 metre in hilly terrain.

Detailed Survey

- Temporary bench marks are fixed at intervals of about 250 m and at all drainage and under pass structures. Levels along the final centre line should be taken at all staked points.
- Levelling work is of great importance as the vertical alignment, earth work calculations

and drainage details are to be worked out from the level notes.

- The cross-section levels are taken up to the desired width, at intervals of 50 to 100 m in plain terrain, 50 to 75 m in rolling terrain, 50 m in built-up areas and 20 m in hilly terrain.
- The cross sections may be taken at closer intervals at horizontal curves and where there is abrupt change in cross slopes.
- All river crossing, valleys etc. should be surveyed in detail up to considerable distances on either side.
- All topographical details are noted down and also plotted using conventional signs. Adequate hydrological details are also collected and recorded.
- A detailed soil survey is carried out to enable drawing of the soil profile. The depth up to which soil sampling is to be done may be 1.5 to 3.0 m below the ground line or finished grade line of the road whichever is lower. However, in case of high embankments, the depth should be up to twice the height of the finished embankment. The spacing of auger borings very much depends upon the soil type and its variations.
- CBR value of soils along the alignment may be determined for designing the pavement.
- The data during the detailed survey should be elaborate and complete for preparing detailed plans, design and estimates of the project.

HIGHWAY PROJECTS

General

In a new highway project, the engineer has to plan, design and construct either a net-work of new roads or a road link. There are also projects requiring re-design and re-alignment of existing roads of upgrading the geometric design standards.

Once a highway is constructed, development takes place along the adjoining land and subsequent changes in alignment or improvements in geometric standards become very difficult. A badly aligned highway is not only a source of potential traffic hazard, but also causes a considerable increase in transportation cost and strain on the drivers and the passengers. Therefore, proper investigation and planning are most important in a road project, keeping in view the present day needs as well as the future developments of the region.

New Highway Project

The new highway project work may be divided into the following stages:

a) Selection of route, finalisation of highway alignment and geometric design details
b) Collection of materials and testing of subgrade soil and other construction materials, mix design of pavement materials and design details of pavement layers
c) Construction stages including quality control.

Route Selection

The selection of route is made keeping in view the requirements of alignment and the geological, topographical and other features of the locality. However special care should be taken as regards the geometric design standards of the road for possible upgrading of speed standards in future, without being necessary to realign the road. After the alignment if finalised, the plans and working drawings are prepared.

Materials and Design

The soil samples collected from the selected route during the soil surveys are tested in the laboratory in order to design the required pavement thickness and the design of embankment and cut slopes. The basic construction materials such as selected soil, aggregates etc. are collected from the nearest borrow pits and quarries and stacked along the road alignment after subjecting these materials to the specified laboratory tests. In order to design the mixes for the pavement component layers and to specify quality control test values during road construction, mix design tests are carried out in the laboratory.

The possibility of using low-cost construction material like soil-aggregate mixes, soft aggregates, stabilized soil and pozzolonic concrete mixes, in the sub-base or base course layers of pavement should be fully explored. When high quality pavement materials like bituminous mixes or cement concrete are to be used in the surface course, the mix design specification and construction control tests should be strictly followed. The pavement thickness is designed based on anticipated traffic, stability and drainage conditions of the subgrade and the type and thickness of pavement layers chosen for the construction.

In India, the **CBR method** has been recommended by the Indian Roads Congress for designing the thickness of flexible pavements.

Construction

The construction of the road may be divided into two stages as follows

1) Earth Work
2) Pavement Construction.

Earth Work

It consists of excavation and construction of the embankments. During the excavation for highway cuts, the earth slopes, their protection and construction of drainage network are taken care of. Highway embankments may be best constructed by rolled-fill method by compacting the soil in layers under controlled moisture and density using suitable rollers. In the case of high embankments, the stability of the embankment foundation and slopes and the possible settlement of the embankment with time are to be investigated.

Pavement Construction

It is subsequently taken up starting with the preparation of subgrade and the construction of sub-base, base and surface courses of the pavement.

Steps in a new project work

The various steps in a new highway project may be summarised as given below:

- **Map Study:** This is carried out with the help of available topographic maps of the area **Reconnaissance Survey:** During reconnaissance survey, a general idea of a topography and other features, field identification of soils and survey of construction materials, by an on-the spot inspection of the site.
- **Preliminary Survey:** Topographic details and soil survey along alternate alignments, consideration of geometric design and other requirements of alignment, preparation of plans and comparison of alternate routes; economic analysis and selection of final alignment.
- **Location of Final Alignment:** Transfer of the alignment from the drawings to the ground by driving pegs along the centre line of finally chosen alignment, setting out geometric design elements by location of tangent points, apex points, circular and transition curves, elevation of centre line and super elevation details.
- **Detailed Survey:** Survey of the highway construction work or the preparation of longitudinal and cross sections, computations of earth work quantities and other construction material and checking details of geometric design elements.
- **Materials Survey**: Survey of construction materials, their collection and testing.
- **Design:** Design details of embankment and cut slopes, foundation of embankments and bridges, and pavement layers and cross drainage structures.
- **Earth Work:** Excavations for highway cutting and drainage system, construction of embankments.
- **Pavement Construction:** Preparation of subgrade, construction of sub-base, base and surface courses.
- **Construction Controls:** Quality control tests during different stages of construction and check for finished road surface such as unevenness, camber, super elevation and extra widening of pavements at curves.
- **Construction Planning and Programming:** The construction planning and programming to be carried out taking into accounts all the restraints and existing problems. In order to minimise the construction cost and time, it is essential to resort to appropriate approaches such as use of Critical Path Method (CPM) and Project Evaluation and Review Technique (PERT).

Re-Alignment Project

Necessity of Re-Alignment

Most of the present highways in India have been upgraded in stages, from the existing local roads of the pre-automobile era. As these roads were then meant for slow traffic, they are found deficient in the geometric design elements for the present-day automobile traffic. There are several stretches of NH in the country having single lane carriageway, narrow bridges and culverts and many locations with sharp horizontal curves and avoidable zigzags, steep gradients and inadequate sight distances. These defects are to be rectified as early as possible at least in stages, starting with roads of greater importance like NH and SH’s. It will be worth-while to adopt more liberal values of geometric design parameters than the ruling minimum values specified, where the conditions are favourable and the costs involved are not excessive. In such cases, it would be possible to upgrade the highway if necessary in future by increasing the width standards only, but without the necessity of re-aligning the road. However, in constrained situations and in difficult terrain, it may not always be economical to improve the existing highway geometries to the recommended design standards. In such cases appropriated speed restrictions have to be imposed to minimise road accidents.

It has been decided as a policy that NH’s should as far as possible be able to fully cater to the traffic moving at design speed, fulfilling the comfort and safety requirements, both for the present and future traffic needs. To achieve this objective, it is necessary to plan improvements in the geometries of roads wherever deficient, to the extent economically practicable along with other improvements such as raising of the road above flood water level, pavement resurfacing or construction of overlay for strengthening the pavement structure.

Types of Improvement

The following types of improvement in alignment of existing road may be carried out:

1) Improvement of horizontal alignment design elements such as, radius, super elevation, transition curve, providing adequate clearance on inner side of the curve or shifting the curve to provide adequate sight distance, elimination of reverse curve and undesirable zigzags, etc.
2) Improvement of vertical alignment design elements like steep gradients, changes in summit curves to increase sight distance, correction of undesirable undulations like humps any dips, etc.

3) Raising the level of a portion of a road which is subjected to flooding, submergence or water-logging during monsoons.
4) Re-construction of weak and narrow bridges and culverts and changes in waterway at locations slightly away from the existing site.
5) Construction of over-bridges or under-bridges at suitable locations across a railway line in place of level crossing or across another road to provide grade separated intersections.
6) Re-alignment required due to a portion of the road being submerged under water at the reservoir area on account of construction of a new dam.
7) Construction of a bypass to avoid the road running through a town or city
8) Defence requirements.

General Principles of Re-Alignment

- While improving the horizontal alignment of roads, improvement in sharp curves and zigzags should be done after considering the whole alignment and not on piece meal basis. The improvement of transition curves would not generally be very costly and therefore such deficiencies should be rectified where-ever necessary. The sight distance available generally gets increased when the horizontal alignment is improved, otherwise the setback distance may be increased at horizontal curves by removing or shifting the obstruction from the inner side of the curve, up to the desired extent,
- While improving the vertical alignment, attempts should be made to provide Overtaking Sight Distance (OSD) at summit curves. On divided highways, the overtaking distance required will be lesser than on un-divided two-way roads, as there is no need to provide for the on-coming vehicles during overtaking operations. However, if it is not possible to provide for OSD, at- least the safe stopping sight distance should be available for the design speed at all locations of the road
- The corrections of minor undulations such as humps and dips may not involve high cost and so it is desirable to provide suitable vertical transition curves for shock-free movement of vehicles travelling at the design speed. Valley curves may be checked for comfort condition and for visibility under the head lights of the vehicles during night driving.
- The road stretches which remain submerged under water even for a short duration of the year or those which are in water-logged areas should be raised before strengthening

or widening pavement section. The formation level be raised such that the subgrade is at least 0.6 m above the HFL. Suitable measures should be adopted against water-logging and care should be taken to provide suitable drainage facilities including the cross drainage works.

- While reconstructing bridges of length greater than 60 m on sites other than the existing ones, separate surveys should be carried out for the selection of suitable sites. The selection of site for major bridges would be governed by the river training works, sub-soil conditions for foundation and hydraulic considerations. However, in small bridges, the road alignment would essentially govern the bridge site selection
- The deciding factor which is being considered for providing over-bridges or under bridges for a NH across railway level crossings is product of number of gate closures and the intensity of traffic on the highway in tonnes per day in the design year. When this product exceeds 50,000 or when the level crossing is within the shunting limits of a railway station, the grade separation is justified. The location is decided keeping in view the highway alignment, the topographic and other site conditions,
- The necessity to provide alternate routes to bypass through traffic is assessed from the origin and destination studies. If the by-passable traffic more than the traffic terminating at the town or built-up area then the bypass may be justified.

Steps in The Re-Alignment Project

1) Reconnaissance of the stretch of road to be re-aligned, study of the deficiencies and the possible changes in alignment
2) Survey of existing road, recording the topographic features and all other existing features including drainage conditions. The width of the land to be surveyed depends on the amount of shifting anticipated when the road is re-aligned.
3) Observations of spot levels along the centre line of the road and cross section levels at suitable intervals. The intervals should be taken at closer intervals at horizontal and vertical curves and near cross drainage works.
4) Soil survey along the stretches of land through which the re-aligned road may pass, preparation of typical soil profiles after testing the soil samples in the lab.
5) Finalisation of the design features of re-aligned road stretches
6) Preparation of drawings and Marking out the centre line of re-aligned road while trying to utilise the existing road to the maximum extent possible.

7) Earth-work and preparation of subgrade of the re-alignment road stretches, setting out and construction of new bridges and culverts
8) Checking the geometric design elements of the newly aligned stretches of the road
9) Design and construction of the new highway pavements

Preparation of Drawing for Re-Alignment Project

The drawings for the re-alignment project should show all the existing features of the road as well as all the proposed improvements.

The following drawings would be needed

1) Plan showing existing road, proposed re-alignment, contours and all other features of importance.
2) Longitudinal section showing natural ground elevation, surface of the existing road and the grade line for the re-construction
3) Cross section showing the existing highway and new roadway drawn at 250m intervals on straights, at the beginning and end of transition curves and at the middle of circular curves. Cross sections are drawn at 50m intervals where the new carriageway falls entirely outside the existing one.

HIGHWAY GEOMETRIC DESIGN

INTRODUCTION

The geometric design of highways deals with the dimensions and layout of visible features of the highway such as horizontal and vertical alignments, sight distances and intersections. The designer may be exposed to either plan a new highway network or improve existing highway to meet the requirements of existing and the anticipated traffic.

It is possible to design and construct highway in stages but the geometric elements have to be planned in the initial stages only it will be expensive and difficult to improve it later.

Geometric design of highways deals with following elements

a) Cross-Section Elements
b) Sight Distance Consideration,
c) Horizontal Alignment Details
d) Vertical Alignment Details
e) Intersection Elements

Design Controls and Criteria

Factors affecting the geometric designs are as follows

Design Controls and Criteria

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

(a) Design speed
(b) Topography or terrain
(c) Traffic factors
(d) Design hourly volume and capacity
(e) Environmental and other factors

Design speed

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

Topography

The topography or the terrain conditions influence the geometric design of highway significantly. The terrains are classified based on the general slope of the country across the alignment, as plain, rolling, mountainous and steep terrains. The design standards specified for different classes of roads, are different depending on the terrain classification. Further in hilly terrain, it is necessary to allow for steeper gradients and sharper horizontal curves due to the construction problems.

Traffic Factors

The factors associated with traffic that affect geometric design of road are the vehicular characteristics and human characteristics of road users. It is difficult to decide the design vehicle or standard traffic lane under mixed traffic flow.

Environmental and Other Factors

The environmental factors such as aesthetics, landscaping, air and noise pollution and other local conditions should be given due considerations 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

Pavement Surface depends on the pavement type. The pavement surface type is decided based on the availability of materials and funds, volume and composition of traffic, sub grade, and climatic conditions. The important characteristics of the pavement are

1) Friction Considerations
2) Unevenness
3) Light Reflecting Characteristics
4) Drainage of Surface Water

1) **Friction**

The friction of skid resistance between vehicle tyre and pavement surface is one of the factors determining the operating speed and the minimum distance requires for stopping of vehicles.

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

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

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

Factors Affecting Friction or Skid Resistance

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

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

- Macro-texture of the pavement surface or its relative roughness
- Condition of pavement namely
- Type and condition of tyre
- Speed of vehicle
- Extent of brake application or brake efficiency
- Load and tyre pressure
- Temperature of tyre and pavement

For the calculation purposes, the IRC has recommended the longitudinal friction co efficient values of 0.35 to 0.40 and lateral co efficient values of 0.15. for expressways and NH’s with design speed of 120 and 100kmph it is 0.10 and 0.11

2) Pavement Unevenness

The longitudinal profile of the road pavement has to he even' in order to provide a good riding comfort to fast moving vehicles and to minimise the VOC. Presence of undulations on the pavement surface is called pavement unevenness which results in

- Increase in Discomfort and Fatigue to Road Users
- Increase in Fuel Consumption and Tyre Wear and Increase in VOC
- Reduction in Vehicle Operating Speed and Increase in Accident Rate

The pavement surface should therefore be maintained with minimum possible unevenness or undulations so that the desired speed can be maintained m conformity with other geometric standards Loose road surface increases the resistance to traction and causes increase in fuel consumption.

The unevenness of pavement surface is commonly measured by using a simple equipment called **'Bump Integrator' (BI),** in terms of **Unevenness Index** which is the cumulative measure of vertical undulations of the pavement surface recorded per unit length of the road. Internationally, the riding quality of a pavement surface is quantified in terms of 'roughness' and is expressed as International Roughness Index (IRI) in units of m/km. The relation between the unevenness measured using bump integrator in mm/km and the International Roughness index in m/km is as follows

$$\mathbf{BI = 630\ (IRI)^{1.12}}$$

Undulations of newly laid pavement surface are sometimes measured using a straight edge and wedge scale, in terms of the depth and number of depressions or ruts along and across the pavement. It may be mentioned here that there are several advanced techniques and equipment available now to evaluate the pavement surface condition.

3) Light Reflecting Characteristics

Night visibility depends upon the colour and light reflecting characteristics of the pavement surface. The glare caused by the reflection of head lights is considerably high on wet pavement surface than on the dry pavement.

- Light coloured or white pavement surface give good visibility at night particularly during rains: however white or light colour of pavement surface may produces glare and eye strain during bright sunlight.
- Black top pavement surface on the other hand provides very poor visibility at nights, especially when the surface is wet.

Cross Slope or Camber

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

1) To prevent the entry of surface water into the pavement layers and the subgrade soil through pavement.
2) To prevent the entry of water into the bituminous pavement layers, as continued contact with water causes stripping of bitumen from the aggregates and results in deterioration of the pavement layer
3) To remove the rain water from the pavement surface as quickly as possible and to allow the pavement to get dry soon after the rain.

The rate of camber or cross slope is usually designated by 1 in ‘n’ which means the transverse slope is in ratio 1 vertical to n horizontal.

The required camber of a pavement depends on

a) Type of pavement surface
b) The amount of rainfall

It is desirable not to provide excessive camber or steep cross slope on road pavements. Only the minimum camber needed to drain off surface water may be adopted keeping in view the type of pavement surface and the amount of rainfall in the locality. Too steep cross slope is not desirable because of the following reasons:

- Transverse tilt of vehicles causes uncomfortable side thrust and a drag on the steering wheel of automobiles. Also, the thrust on the wheels along the pavement edges is more causing unequal wear of the tyres as well as road surface
- Discomfort causing throw of vehicle when crossing the crown during overtaking operations.
- Problems of possible toppling over of highly laden bullock carts and trucks
- Formation of cross ruts due to rapid flow of water
- Tendency of most of the vehicles to travel along the centre line

Recommended values of camber

The values of camber recommended by the IRC for different types of road surfaces are given in the below table.

Recommended values of camber for different types of road surfaces

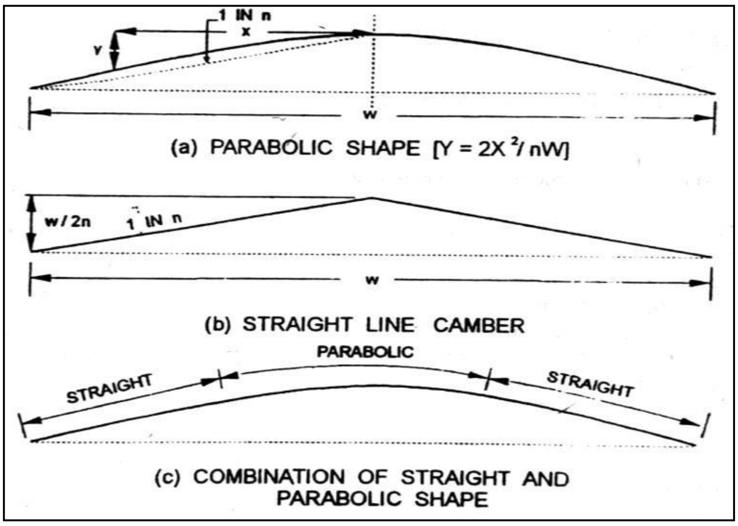
Sl. No.	Type of road surface	Range of camber in areas of	
		Heavy rainfall	Low rainfall
1.	Cement concrete and high type bituminous surface	1 in 50 or 2.0 %	1 in 60 or 1.7%
2.	Thin bituminous surface	1 in 40 or 2.5 %	1 in 50 or 2.0 %
3.	Water bound Macadam and gravel pavement	1 in 33 or 3.0 %	1 in 40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

The cross slope for shoulders should be 0.5% steeper than the cross slope of adjoining pavement, subject to a minimum of 3.0% and a maximum value of 5.0% for earth shoulders. The cross slope suggested for the carriageway, paved shoulders and edge strip of expressways with bituminous surface as well as cement concrete surface is 2.5 % in regions with annual rain fall exceeding 1000 mm and 2.0 % in places with less than 1000 mm rain fall.

SHAPE OF CROSS SLOPE

In the field, camber of the pavement cross section is provided with a suitable shape. Different shapes that are commonly adopted are

1) Parabolic
2) Straight Line
3) Straights with parabolic curve



Providing Camber in the field

In order to provide the desired amount and shape of camber, templates or camber boards are prepared with the chosen shape and specified cross slope and they can be used to check the lateral profile of finished pavements.

a) **Parabolic Camber**

$$y = x^2/a$$

where a = nW/2

b) **Straight Line Camber**

$$y = W/2n$$

W – width, n – cross slope in 1 in n

WIDTH OF PAVEMENT OR CARRIAGEWAY

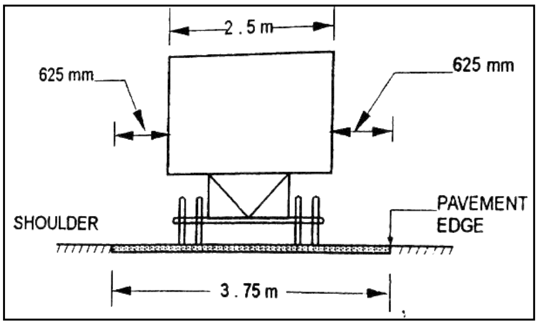
The width of pavement or carriageway depends on

1) Width of Traffic Lane
2) Number of Lanes.

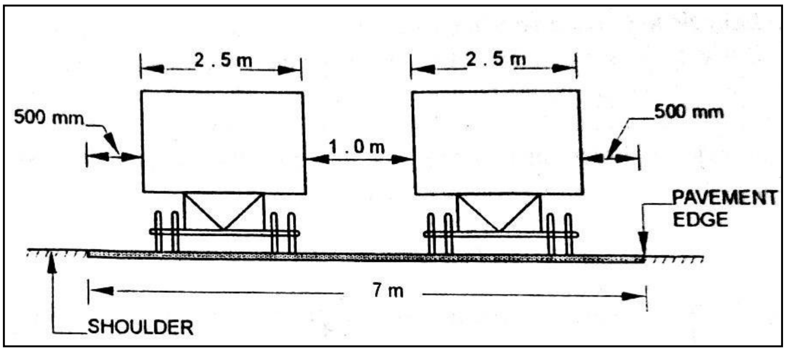
The portion of carriageway width that is intended for one line of traffic movement is called a traffic lane. As different classes of vehicles travel along the same roadway generally the lane width is decided based on a standard vehicle such as the passenger car. However, it is also necessary to consider the maximum width of the largest vehicle class such as the heavy commercial vehicle (HCV) which is legally permitted to use the roadway in the country.

Width of carriageway recommended by IRC

Class of Road	Width of Carriageway, m
Single Lane Road	3.75
Two Lane Road, without raised kerbs	7.0
Two Lane Road, with raised kerbs	7.5
Intermediate Carriageway	5.5
Multi Lane Pavements	3.5 per lane



Single Lane Pavement



Two Lane Pavement

MEDIANS/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 directions. The main function of the median is to prevent head-on collision between vehicles moving in opposite directions on adjacent lanes. The median is also called or traffic separator. The traffic separators used may be in the form of pavement markings, physical dividers or area separators. Pavement marking is the simplest of all these, but this will not rule out head-on collision. The mechanical separator may be suitably designed keeping in view safety considerations.

KERBS

Kerb indicates the boundary between the pavement and median or foot path or island or shoulder. It is desirable to provide kerbs on urban roads. Refer Fig. There are a variety of kerb designs. Kerbs may be mainly divided based on their functions.

a) 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.

b) Semi-Barrier Type Kerbs

When the pedestrian traffic is high, these kerbs are provided. Their height is 15 cm above the pavement edge.

c) 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.

d) Submerged Kerbs

They are used in rural roads. The kerbs are provided at pavement edges between pavement edge and shoulder

ROAD MARGINS

The portion of the road beyond the carriageway and on the roadway can be generally called road margin. Various elements that form the road margins are given below.

Shoulders

Shoulders are provided on both sides of the pavement all along the road in the case of undivided highway and are provided on the outer edge of the highway in divided carriage way

The important functions of shoulders are:

(a) Shoulders provide structural stability and support to the edges of the flexible pavements.

(b) The capacity of the carriageway and the operating speeds of vehicles increase if the shoulders are laid and maintained in good condition.

(c) Shoulders serve as emergency lanes for vehicle compelled to be taken out of the main carriageway or roadway. Shoulders should have sufficient load bearing capacity to support loaded truck even in wet weather

(d) Shoulders also act as service lanes for vehicles that are disabled. The width of shoulder should be adequate to accommodate stationary vehicle fairly away from the edge of adjacent lane.

Guard rails

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

Footpath or side-walk

In order to provide safe facility to pedestrians to walk along the roadway, foot paths or side-walks are provided in urban areas where the pedestrian traffic is noteworthy and the vehicular traffic is also heavy. By providing good foot path facility, the pedestrians can keep off from the carriageway and they are segregated from the moving vehicular traffic. Thus, the operating speeds of the vehicular traffic increases and there will be marked reduction in accidents involving pedestrians.

Drive ways

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

Cycle tracks

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

Parking lanes

Parking lanes are provided on urban roads to allow kerb parking. As far as possible only 'parallel parking' should be allowed as it is safer for moving vehicles. For parallel parking, the minimum lane width should be 3.0 m.

Bus bays

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

Lay-byes

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

Frontage roads

Frontage roads are provided to give access to properties along an important highway with controlled access to express way or freeway. 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 separation.

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. Formation or roadway width is the top width of the highway embankment or the bottom width of highway cutting excluding the side drains.

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 it depends on the importance of the road and possible future development. A minimum land width has been prescribed for each category of road. A desirable range of land width has also been suggested for each category of road. While acquiring land for a highway it is desirable to acquire more width of land as the cost of adjoining land invariably increases as soon as the new highway is constructed.

SIGHT DISTANCE

Sight Distance and Importance

Sight distance is the length of road visible ahead of 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. The sight distance between the driver and the object is measured along the road surface.

Restrictions to Sight Distance

Restrictions to visibility or sight distance may be caused in the following circumstances

- At horizontal curves, when the line of sight is obstructed by objects at the inner side of the curve. Here the sight distance is measured along the centre line of the horizontal curve when the vehicle driver is able to see another vehicle or object on the carriageway
- At a vertical curve, the line of sight is obstructed by the road surface of the summit curve (i.e., a vertical curve of the road with convexity upwards)
- In this case also the sight distance is measured along the centre line of the vertical curve when the vehicle driver is able to see another vehicle or object on the road
- 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, Here the sight distance for each vehicle driver is the distance from the position when the two can see each other up to the intersection point of the two roads.

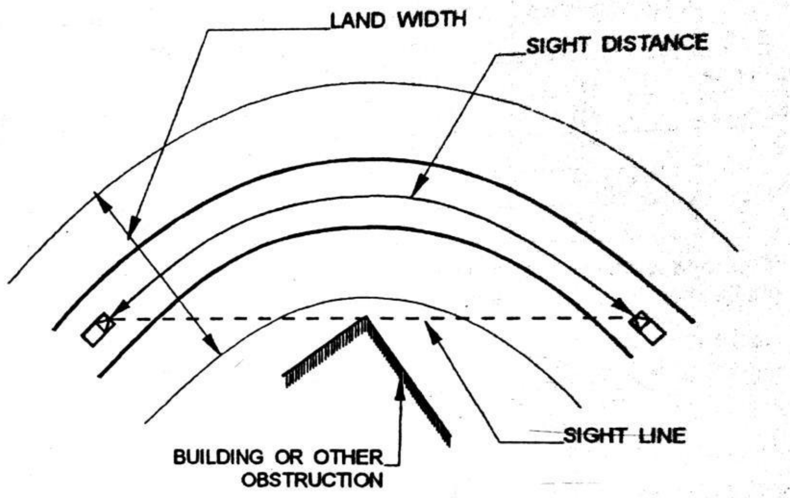
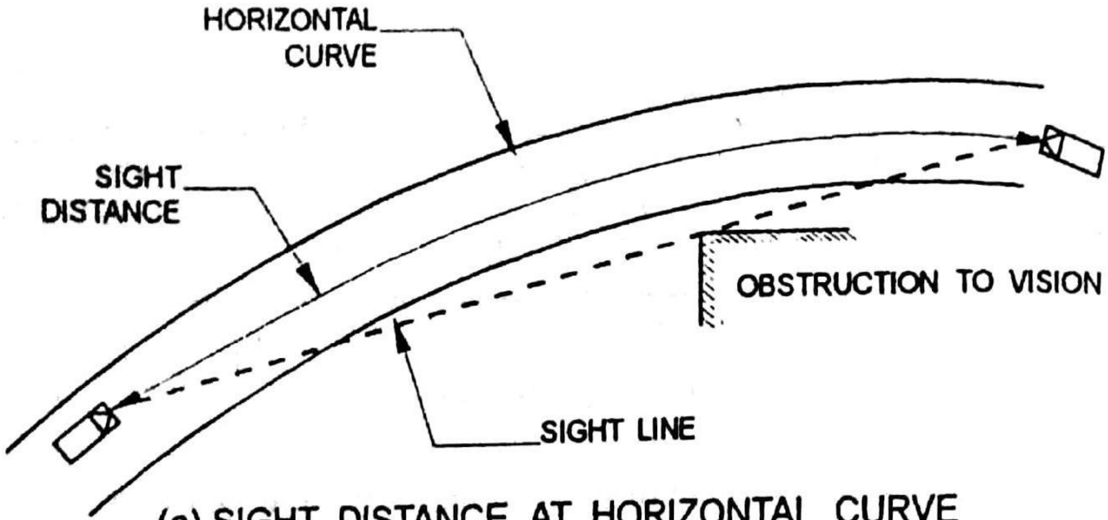
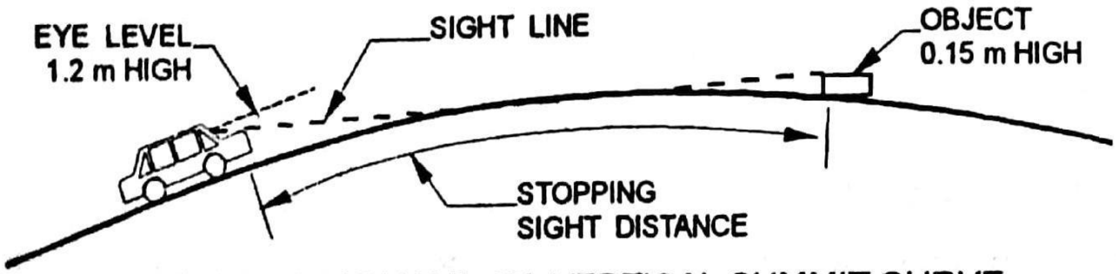


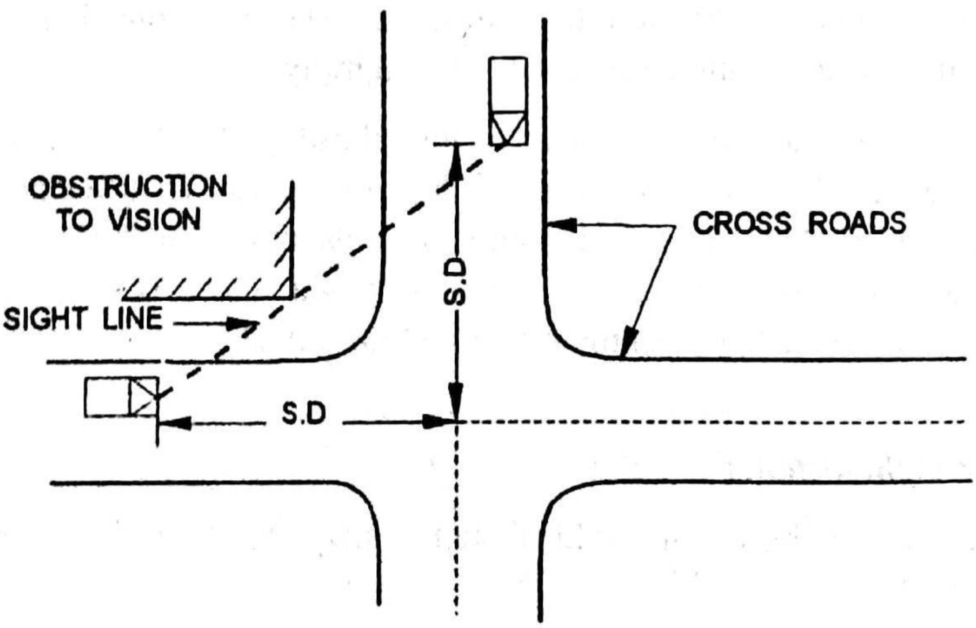
Fig. 4.5 Obstruction to visibility at horizontal curve



(a) SIGHT DISTANCE AT HORIZONTAL CURVE



(b) SIGHT DISTANCE AT VERTICAL SUMMIT CURVE



(c) SIGHT DISTANCE (S.D) AT INTERSECTION

TYPES OF SIGHT DISTANCE

Sight distance required by drivers applies to geometric designs of highways and for traffic control. Three types of sight distances are considered in the design

a) **Stopping Sight Distance (SSD)** or absolute minimum sight distance
b) **Safe Overtaking Sight Distance (OSD)** or Passing Sight Distance
c) **Safe Sight Distance** for entering into uncontrolled intersections.

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

d) **Intermediate Sight Distance**
e) **Head Light Sight Distance**

STOPPING SIGHT DISTANCE (SSD)

Factors on which visibility or sight distance depends

The minimum distance visible to a driver ahead or the sight distance available 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. Therefore, this Stopping Sight Distance (SSD) is also called Absolute Minimum Sight Distance. This is also sometimes called Non-Passing Sight Distance.

The sight distance available to a driver travelling on a road at any instance depends on the following factors:

a) Features of the road ahead
b) Height of the driver's eye above the road surface
c) Height of the object above the road surface

IRC has suggested the height of eye lev el of driver as 1.2 m and the height of the object as 0.15 m above the road surface.

Factors on which stopping distance depends

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

a) Total reaction time of the driver
b) Speed of vehicle
c) Efficiency of Brakes
d) Frictional Resistance between the road and the tyre

e) Gradient of the road, if any

TOTAL REACTION TIME OF DRIVER

Reaction time of the driver is the time taken from the instant the object is visible to the driver to the instant the brakes are effectively applied. The actual time gap or the reaction time of the driver depends on several factors. During this period of time the vehicle travels a certain distance at the original speed, which may be assumed to be the design speed of the road. Thus, the stopping distance increases with increase reaction time of the driver.

The total reaction time (**t**) may be split up into two parts

Perception Time

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

Brake Reaction Time

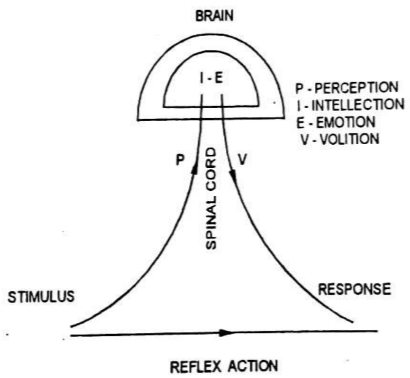
It is also depending on several factors including the skill of the driver, the type of the problems and various other environmental factors.

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, viz., time taken by the driver for

1) Perception
2) Intellection
3) Emotion
4) Volition



The PIEV time of a driver also depends on several factors such as physical and psychological characteristics of the driver, type of the problem involved, environmental conditions and temporary factors.

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.

Efficiency of brakes

The braking efficiency is said to be 100 percent if the wheels are fully locked preventing them from rotating on application of the brakes. This will result in 100 percent skidding which is normally undesirable, except in utmost emergency. Also skidding is considered to be dangerous, as it is not possible for the driver to easily control a vehicle after it starts skidding.

Frictional resistance between road and tyres

The factional resistance developed between road and tyres depends upon the 'skid resistance' or the coefficient of friction, f between the road surface and the tyres of the vehicle.

Analysis of Stopping Distance

The stopping distance of a vehicle is the sum of

a) The distance travelled by the vehicle at uniform speed during the total reaction time, t which is known as **LAG DISTANCE**.
b) The distance travelled by the vehicle after the applications of the brakes, until the vehicles comes to a dead stop which is known as **BRAKING DISTANCE**.

LAG DISTANCE

During the total reaction time, t seconds the vehicle may be assumed to move forward with a uniform speed at which the vehicle has been moving and this speed may be taken as the design speed. If ‘v’ is the design speed in m/sec and ‘t’ is the total reaction time of the driver in seconds, then

Lag Distance = v t

If the design speed is V kmph, then the lag distance = $V\ t \times \frac{1000}{60 \times 60}$

$= 0.278\ V\ t \approx \mathbf{0.28\ V\ t}$ in meters

IRC has recommended the value of reaction time t as 2.5 sec for calculation of Stopping Distance

BRAKING DISTANCE ON LEVEL SURFACE

The coefficient of friction f depends on several factors such as the type and condition of the pavement and the value of f decreases with the increase in speed. IRC has recommended a set of friction co efficient values for the determination of stopping sight distance.

Speed, kmph	20 – 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

The braking distance, $\mathbf{l} = \frac{v^2}{2gf}$

Where l - braking distance, m

v - speed of the vehicle, m/sec

f - design coefficient of friction, f (0.40 to 0.35)

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

STOPPING DISTANCE ON LEVEL ROAD

Stopping Distance, SD = Lag Distance + Braking Distance

$$\mathbf{SD = v\ t +} \frac{v^2}{2gf} \text{ in meters}$$

If speed is V kmph, stopping distance

$$\mathbf{SD = 0.278\ V\ t +} \frac{V^2}{254f} \text{ in meters}$$

STOPPING DISTANCE AT SLOPES

$$\mathbf{SD = [v\ t +} \frac{v^2}{2g(f \pm 0.01n)} \mathbf{]} \text{ in meters}$$

If speed is V kmph, stopping distance

$$\mathbf{SD = 0.278\ V\ t +} \frac{V^2}{254(f \pm 0.01n)} \text{ in meters}$$

IRC has recommended the SSD values for different speed as follows

Design Speed, kmph	20	25	30	40	50	60	65	80	100
SSD for design, m	20	25	30	45	60	80	90	120	180

PROBLEMS

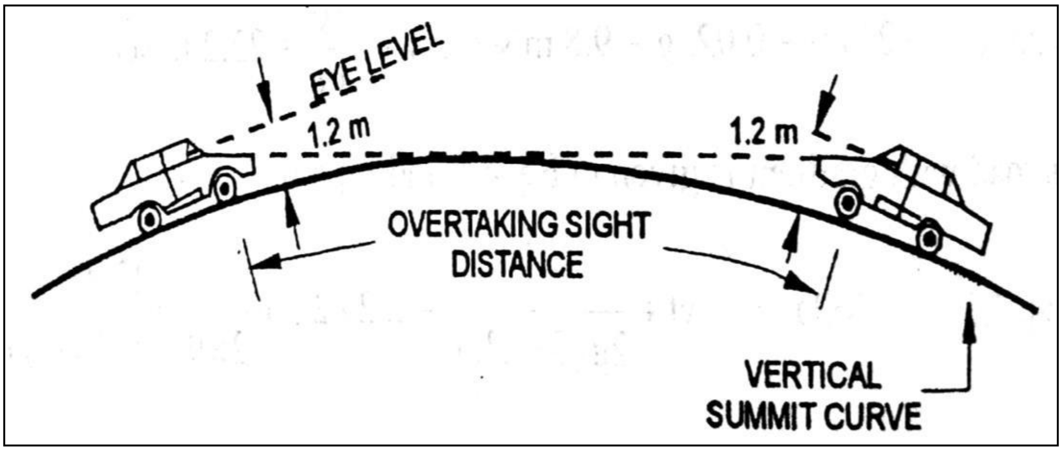
OVERTAKING SIGHT DISTANCE (OSD)

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**

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



Measurement of OSD

Factor Affecting OSD

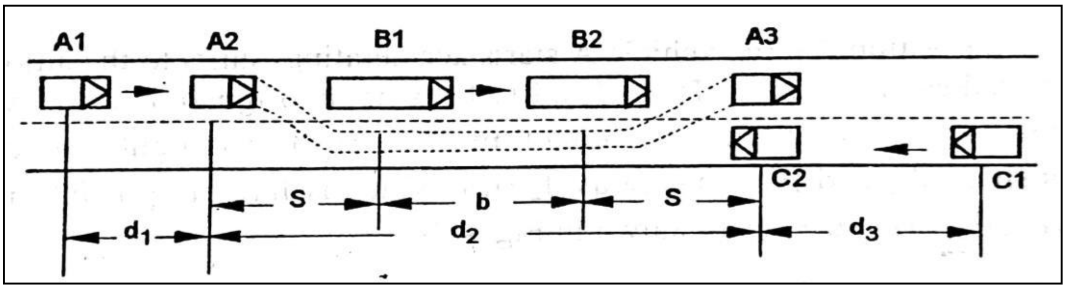
- Speeds of
    1) Overtaking Vehicle
    2) Overtaken Vehicle
    3) Vehicle coming from opposite direction
- Distance between the overtaking and overtaken vehicles, the minimum spacing between vehicles depends on the speeds
- Skill and reaction time of the driver
- Rate of acceleration of overtaking 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 d1, d2 and d3.

- d1 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
- d2 is the distance travelled (m) travelled by the vehicle A during the actual overtaking operation during T (secs) from position A2 to A3
- d3 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 = d1 + d2 + d3 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 Vb (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 A_2 to A3
- The distance d_2 is split up into three parts

a) Spacing, s (m) between A_2 and B

b) Distance b (m) travelled by the slow vehicle B between B_1 and B_2 during the overtaking manoeuvre of A

c) Spacing (m) between B_2 and A_3

- During this overtaking time T (sec), the vehicle C coming from opposite direction travels through a distance d_3 from position C_1 to C_2

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 vb (m/sec) during the reaction time, t (sec) = vb 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

$$\mathbf{d_1 = 2v_b}$$

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 d2 (m), which is further split into three parts, viz.,

$$\mathbf{d_2 = (s + b + s)}$$

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

$$\mathbf{s = (0.7\ vb + 6)}$$

d) Now the time T depends on speed of overtaken vehicle B and the average acceleration a (m/sec^2) of overtaking vehicle A. The overtaking time T (sec) may be calculated by equating the distance d2 to (vb T + ½ a T^2) using the general formula for the distance travelled by a uniformly accelerating body with initial speed vb m/sec and a is the average acceleration during overtaking in m/sec^2

$$\mathbf{d_2 = (vb\ T + 2s)}$$

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_2 (m) between positions C_1 to C_2. Hence,

$$\mathbf{d_3 = v\,T\ (m)}$$

In m/sec units

$$\mathbf{OSD = (d_1 + d_2 + d_3) = (v_b\,t + v_b\,T + 2s + vT)}$$

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

$$T = \sqrt{\frac{4\,s}{a}}$$

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

a= average acceleration during overtaking, m/sec.

In kmph units

$$\mathbf{OSD = 0.28\,V_b\,t + 0.28\,V_b\,T + 2s + 0.28\,V\,T}$$

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

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

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,

$\mathbf{v_b = (v - 4.5)}$ **m/sec**

$\mathbf{V_b = (V - 16)}$ **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

Overtaking sight distance on two-lane highways for different speeds

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

CRITERIA FOR SIGHT DISTANCE REQUIREMENT ON HIGHWAY

Absolute Minimum Sight Distance

SSD for the design speed is the absolute mining sight distance and this should be made available all along the road stretch 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 specify the speed limit along with appropriate warning signs.

Overtaking Sight Distance

It is desirable that adequate overtaking sight distance is available on most of the road stretches such that the vehicles travelling at the design speed can overtake slow vehicles at the earliest opportunity.

- On road stretches with two-way traffic movement, the minimum overtaking distance should be **($d_1 + d_2 + d_3$)** where overtaking is not prohibited.
- On divided highways and on roads with one way traffic regulation, the overtaking distance need be only **($d_1 + d_2$)** as no vehicle is expected from the opposite direction.
- On divided highways with four or more lanes, it is not essential to provide the usual OSD; however, the sight distance on any highway should be more than the SSD, which is the absolute minimum sight distance.

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 sufficient for safe overtaking. Sign posts should be installed at sufficient distance m 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**

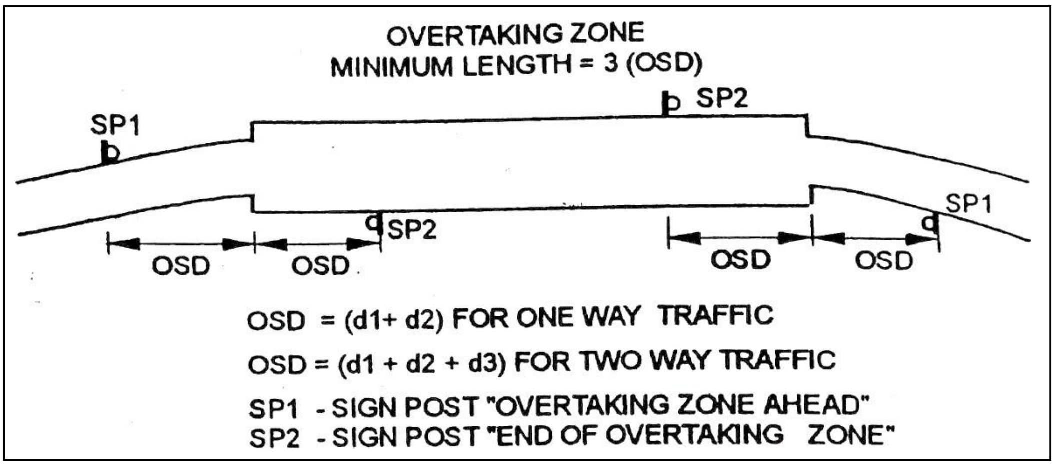
The minimum length of overtaking zone = 3 (OSD)

The desirable length of overtaking zones = 5 (OSD)

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



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. This is all the more important at uncontrolled intersections. The sight line is obstructed by structures or other objects at the corners of the intersections. The area of unobstructed sight formed by the lines of vision is called the sight triangle.

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

a) Enabling the approaching vehicle to change speed
b) Enabling approaching vehicle to stop
c) Enabling stopped vehicle to cross a main road
d) Enabling the approaching vehicle to change speed

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 m 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 Kmph

Horizontal Curves

A horizontal highway curve is a curve in plan to provide change in direction to the centre line of a road. A simple circular curve may be designated by either the radius, R of the curve in meters or the degree, D of the curve. The degree of the curve (D°) is the central angle subtended by an arc of length 30 m and is given by the relation, RDπ/180 = 30. Therefore, the relation between the radius and degree of the circular curve is given by, R = 1720 / D

When a vehicle traverses a horizontal curve, the centrifugal force acts horizontally outwards through the centre of gravity of the vehicle. The centrifugal force developed depends on the radius of the horizontal curve and the speed of the vehicle negotiating the curve. This centrifugal force is counteracted by the transverse frictional resistance developed between the tyres and the pavement which enables the vehicle to change the direction along the curve and to maintain the stability of the vehicle. Centrifugal force P is given by the equation:

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

Where

P = centrifugal force, kg

W = weight of the vehicle, kg

R = radius of the circular curve, m

v = speed of vehicle, m/sec

g = acceleration due to gravity = 9.8 m/sec

The ratio of the centrifugal force to the weight of the vehicle, P/W is known as the 'centrifugal ratio' or the 'impact factor'. Therefore, centrifugal ratio

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

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

1) Tendency to overturn the vehicle outwards about the outer wheels
2) Tendency to skid the vehicle laterally, outwards

Overturning Effect

The overturning moment due to centrifugal force, P = P x h

This is resisted by the restoring moment due to weight of the vehicle W and is equal to (Wb/2)

The equilibrium condition for overturning will occur when [illegible] = [illegible] or [illegible] = [illegible]

overturning will occur

And for safety [illegible] > [illegible]

Transverse Skidding Effect

The centrifugal force developed has the tendency to push the vehicle outwards in the transverse direction.

The equilibrium condition for the transverse skid resistance developed is given by

$$\mathbf{F = F_A + F_B}$$
$$\mathbf{= f\,(R_A + R_B)}$$
$$\mathbf{= f\,W}$$

Where f = coefficient of friction between the tyre and the pavement surface in the transverse direction

R_A, R_B = Normal Reactions at the wheels A and B

W = weight of the vehicle

When the centrifugal ratio [illegible] = [illegible] = [illegible] skidding takes place

For safety [illegible] > [illegible]

Thus, to avoid both overturning and lateral skidding on a horizontal curve, the

[illegible] < [illegible]

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.**

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

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

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 θ

f = design value of lateral friction coefficient = 0.15

v = speed of the vehicle, m/sec

R = radius of the horizontal curve, m

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

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

$$e + f = \frac{(0.2778V)^2}{9.81R} = \frac{V^2}{127R}$$

The maximum value of super elevation is liBRCEd 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

$$V = \sqrt{127fR}$$

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

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

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

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

$$f = \left(\frac{v^2}{gR} - 0.07\right) \text{ or } \left(\frac{V^2}{127R} - 0.07\right)$$

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 cm[1]' is calculated by considering the. design coefficient of lateral friction and the maximum super elevation. The safe allowable speed

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

or

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

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 liBRCEd 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:

(a) Elimination of crown of the cambered section

(b) Rotation of pavement to attain full superelevation

RADIUS OF HORIZONTAL CURVE

Horizontal curves of highways are generally designed for the specified ruling design speed of the highway. However, if this is not possible due to site restrictions, the horizontal curves may be designed considering the specified minimum design speed of the highway.

For a particular speed of vehicle, the centrifugal force is dependent on the radius of the horizontal curve. To keep the centrifugal ratio **P/W or v^2 /g R** within a low limit, the radius of the horizontal curve should be kept correspondingly high. The centrifugal force, P developed due to a vehicle negotiating a horizontal curve of radius, R at a speed, v m/sec or V kmph is counteracted by the superelevation, e and lateral friction coefficient, f.

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

Also

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

The minimum design speed is V' Kmph, the absolute minimum radius of horizontal curve

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

v and V – ruling speeds in m/sec and Kmph

V' – minimum design speed in kmph

e - rate of superelevation, (0.07)

f – co efficient of friction 0.15

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

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, W_e 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.

Mechanical Widening

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

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

Psychological Widening

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

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

Hence Total Widening We is given by We = Wm + Wps

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

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 full 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

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 Rm. Hence the rate of change of centrifugal acceleration is distributed over a length Ls

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

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

The IRC has recommended the following equation

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

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

The length of the transition curve Ls is given by

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

If the design speed is given in kmph

$$L_s = \frac{V^3}{46.7CR}$$

C - rate of change of centrifugal acceleration, m/sec^3

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 +We)

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

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

b) For mountainous and steep terrain

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

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 Ls and the radius of the circular curve is R, the shift S of the transition curve is given by the formula

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

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, Lc which may be greater or lesser than S

a) When Lc > S

When the length of curve Lc 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 id given by

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

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 Lc < S

If the length of the curve Lc 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 Lc and the setback distance m' is worked out in 2 parts

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

The setback distance is given by

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

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 θ 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 α = T (1 – cos α)

DESIGN OF VERTICAL ALIGNMENT

INTRODUCTION

The natural ground or the topography may be level at some places, but may have slopes of varying magnitudes at other locations. While aligning a highway it is the common practice to follow the general topography or profile of the land, keeping in view the drainage and other requirements on each stretch. This is particularly with a view to minimise deep cuttings and very high embankments. Hence the vertical profile of a road would have level stretches as well as slopes or grades.

In order to have smooth vehicle movements on the roads, the changes in the gradient should be smoothened out by the vertical curves. The vertical alignment is the elevation or profile of the centre line of the road. The vertical alignment consists of grades and vertical curves.

The vertical alignment of a highway influences

1) Vehicle Speed
2) Acceleration and Deceleration
3) Stopping Distance
4) Sight Distance
5) Comfort While Travelling at High Speeds
6) Vehicle Operation Cost.

Gradient

Gradient is the rate of rise or fall along the length of the road with respect to the horizontal. It is expressed as a ratio of 1 in x (1 vertical unit to x horizontal units). The gradient is also expressed as percentages such as n%, the slope being n vertical units to 100 horizontal units

Types of gradient

a) Ruling Gradient
b) Limiting Gradient
c) Exceptional Gradient
d) Minimum Gradient

Ruling gradient

The ruling gradient or the design gradient is the maximum gradient with which the designer attempts to design the vertical profile of the road. This depends on the terrain, length of the grade, speed, pulling power of the vehicle and the presence of the horizontal curve. In plain terrain, it may be possible to provide at gradients, but in hilly terrain it is not economical and sometimes not possible also.

The IRC has recommended ruling gradient values of

a) 1 in 30 on plain and rolling terrain
b) 1 in 20 on mountainous terrain
c) 1 in 16.7 on steep terrain.

Limiting gradient

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

Exceptional gradient

In some extra ordinary situations, it may be unavoidable to provide still steeper gradients than limiting gradient at least for short stretches and in such cases the steeper gradient up to 'exceptional gradient' may be provided. However, the exceptional gradient should be strictly liBRCEd only for short stretches not exceeding about 100 m at a stretch.

Minimum gradient

This is important only at locations where surface drainage is important. Camber will take care of the lateral drainage. But the longitudinal drainage along the side drains requires some slope for smooth flow of water.

The road with zero gradient passing through level land and open side drains are provided with a gradient of 1 in 400.A minimum of 1 in 500 may be sufficient to drain water in concrete drains or gutter, on inferior surface of drains 1 in 200 or 0.5%, on kutcha open drains steeper slope up to 1 in 100 or 1 % may be provided

Gradient for roads in different terrains

Type of Terrain	**Ruling Gradient**	**Limiting Gradient**	**Exceptional Gradient**
Plain or Rolling	3.3 %, 1 in 30	5 %, 1 in 20	6.7 %, 1 in 15
Mountainous terrain and steep terrain having elevation more than 3000 m above the mean sea level	5 %, 1 in 20	6 %, 1 in 16.7	7 %, 1 in 14.3
Steep terrain up to 3000 m height above mean sea level	6 %, 1 in 16.7	7 %, 1 in 14.3	8 %, 1 in 12.5

Grade Compensation on Horizontal Curve

When sharp horizontal curve is to be introduced on a road which has already the maximum permissible gradient, then the gradient should be decreased to compensate for the loss of tractive effort due to curve. This reduction in gradient at the horizontal curve is called Grade compensation or compensation in gradiebt at the horizontal curve, which is intended to off-set the extra tractive effort involved at the curve. This is calculated from the below equation

$$\text{Grade Compensation \%} = \frac{\square\square + \square}{\square}$$

The max value of grade compensation is liBRCEd to 75/R, where R is the radius of the circular curve in m

As per IRC the grade compensation is not necessary for gradients flatter than 4.0 %, and therefore when applying grade compensation correction, the gradients need not be eased beyond 4 %. **The compensated gradient = Ruling Gradient – Grade Compensation**

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:

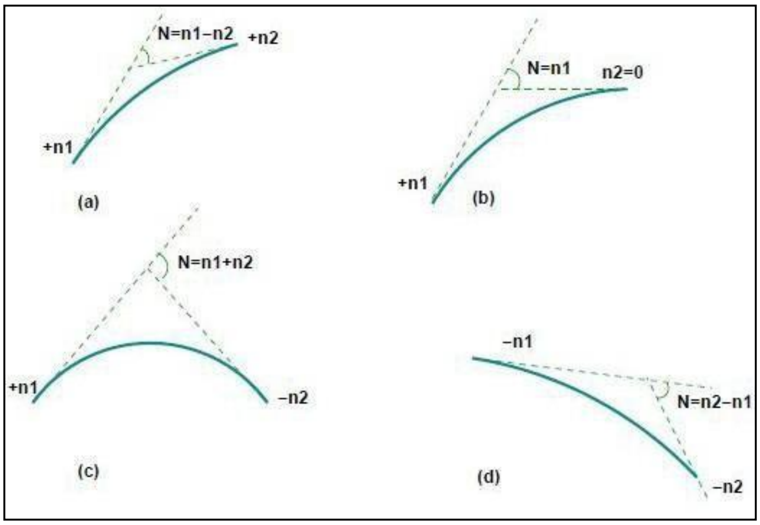
(a) Summit curves or crest curves with convexity upwards

(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

a) When a positive gradient meets another positive gradient
b) When positive gradient meets a at gradient
c) When an ascending gradient meets a descending gradient.
d) When a descending gradient meets another descending gradient



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 an ascending gradient, (+ n1) meets with a descending gradient, (- n2).

Therefore, deviation angle, N= n1 - (- n2) = (n1 + n2)

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

Type of Summit Curve

Many curve forms can be used with satisfactory results; the common practice has been to use parabolic curves in summit curves. This is primarily because of the ease with it can be laid out as well as allowing a comfortable transition from one gradient to another.

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

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

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

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

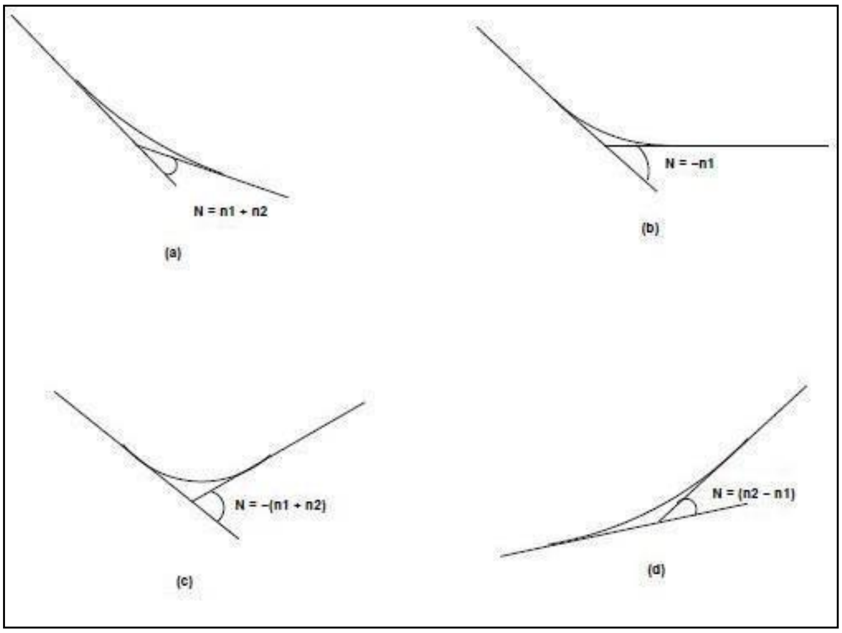
Valley curve

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, (- n1) meets with an ascending gradient, (+ n2).

Therefore, deviation angle, N= - n1 - (+ n2) = - (n1 + n2)

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

Length of transition curve for Comfort condition

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

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

Where

L – Total length of valley curve = 2Ls

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

C – the allowable rate of change of centrifugal acceleration, the value of C may be taken as $0.6 m/sec^3$

v – Design speed in m/s

V – design speed in kmph

The minimum radius of cubic parabolic valley curve is given by $R = \frac{L_s}{N} = \frac{L}{2N}$

Length of the summit curve for OSD or ISD

a) When L > OSD or ISD

If the valley curve is assumed to be parabolic shape, with equation $y = a x^2$, where $a = N/2L$

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

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

Where

h1 – the average height of head light = 0.75m

α - 1^o, the beam angle

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

L – Total length of valley curve, m

S – OSD or ISD, m

N - Deviation angle = (n1 + n2), with slopes – n1 and + n2

b) When L < OSD or ISD

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

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

Where

h1 – the average height of head light = 0.75m

α - 1º, the beam angle

$$L = 2S - \frac{(1.5 + 0.035S)}{N}$$

MODULE – 3

PAVEMENT MATERIALS

INTRODUCTION

Subgrade Soil

Subgrade soil is an integral part of the road pavement structure which directly receives the traffic load from the pavement layers. The subgrade soil and its properties are important in the design of pavement structure. The main function of the subgrade is to give adequate support to the pavement and for this the subgrade should possess sufficient stability under adverse climate and loading conditions.

The formation of waves, corrugations, rutting and shoving in black top pavements and the phenomena of pumping, blowing and consequent cracking of cement concrete pavements are generally attributed due to the poor subgrade conditions.

Desirable Properties

The desirable properties of soil as a highway material are

a) Stability
b) Incompressibility
c) Permanency of strength
d) Minimum changes in volume and stability under adverse conditions of weather and ground water
e) Good drainage
f) Ease of compaction

The soil should possess adequate stability or resistance to permanent deformation under loads, and should possess resistance to weathering, thus retaining the desired subgrade support. Minimum variation in volume will ensure minimum variation in differential strength values of the subgrade. Good drainage is essential to avoid excessive moisture retention and to reduce the potential frost action. Ease of compaction ensures higher dry density and strength under particular type and amount of compaction.

SOIL CLASSIFICATION

Soil Classification Based on Grain Size

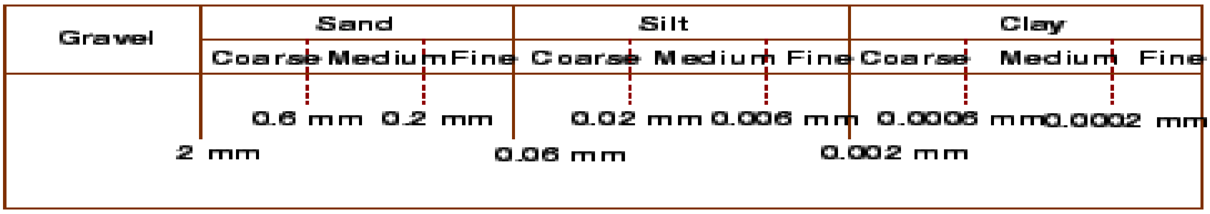
There are several soil classification systems based on grain size of soil, according to which soils have been classified as

a) Gravel

b) Sand

c) Silt and Clay

The most widely accepted grain size classification system is MIT soil classification system. The Bureau of Indian Standards (BIS) has also adopted the same limits as MIT system for the Indian Standard Classification System for soil grains. The limits of grain size are as follows.



Very coarse soils	Boulder size		> 300 mm
	Cobble size		80 - 300 mm
Coarse soils	Gravel size (G)	Coarse	20 - 80 mm
		Fine	4.75 - 20 mm
	Sand size (S)	Coarse	2 - 4.75 mm
		Medium	0.425 - 2 mm
		Fine	0.075 - 0.425 mm
Fine soils	Silt size (M)		0.002 - 0.075 mm
	Clay size (C)		< 0.002 mm

Highway Research Board (HRB) classification of soils

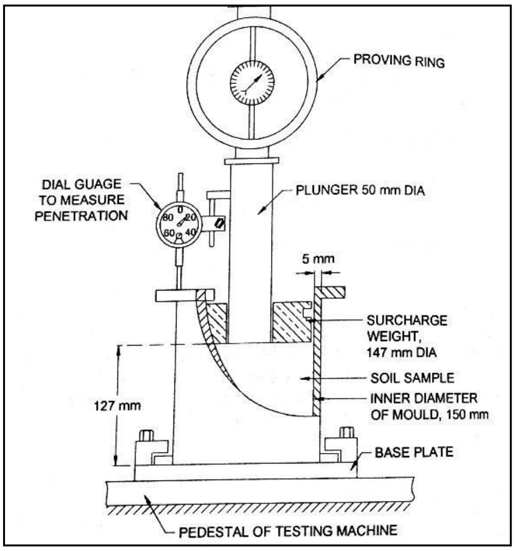
The Highway Research Board (HRB) soil classification method is also called Revised Public Roads Administration (PRA) soil classification system. With just three simple laboratory tests namely sieve analysis, liquid limit and plastic limit, it is possible to classify the soils. The HRB soil classification system is generally adopted in highway engineering for the classification of subgrade soils.

Soils are divided into seven groups A-1 to A-7. A-1, A-2 and A-3 soils are granular soils, percentage fines passing 0.075 mm sieve being less than 35. A-4, A-5, A-6 and A-7, soils are fine grained or silt-clay soils, passing 0.075 mm sieve being greater than 35 percent.

- A-1 soils are well graded mixture of stone fragments, gravel coarse sand, fine sand and non-plastic or slightly plastic soil binder. The soils of this group are subdivided into two subgroups, A- 1-a, consisting predominantly of stone fragments or gravel and A-I-b consisting predominantly of coarse sand.
- A-2 group of soils include a wide range of granular soils ranging from A- 1 to A-3 groups, consisting of granular soils and up to 35% fines of A-4, A-5, A-6 or A-7 groups. Based on the fines content, the soils of A-2 groups are subdivided into subgroups A-2-4, A-2- 5, A-2-6 and A-2-7.
- A-3 soils consist mainly, uniformly graded medium or fine sand similar to beach sand or desert blown sand. Stream-deposited mixtures of poorly graded fine sand with some coarse sand and gravel are also included in this group.
- A-4 soils are generally silty soils, non-plastic or moderately plastic in nature with liquid limit and plasticity index values less than 40 and 10 respectively
- A-5 soils are also silty soils with plasticity index less than 10%, but with liquid limit values exceeding 40%. These include highly elastic or compressible, soils, usually of diatomaceous of micaceous character.
- A-6 group of soils are plastic clays, having high values of plasticity index exceeding 10% and low values of liquid limit below 40%; they have high volume change properties with variation in moisture content.
- A-7 soils are also clayey soils as A-6 soils, but with high values of both liquid limit and plasticity index, (LL greater than 40% and P1 greater than 10%). These soils have low permeability and high volume change properties with changes in moisture content.

CALIFORNIA BEARING RATIO (CBR) TEST

This is a penetration test developed by the California division of highway. For evaluating the stability of soil subgrade and other pavement materials. The test results have been correlated with flexible pavement thickness requirement for highway and airfield. CBR test may be conducted in the laboratory on a prepared specimen in a mould or in situ in the field.



CBR Test Set Up

Laboratory CBR test

The laboratory CBR apparatus consists of

Cylindrical mould

Mould 150mm dia, 175mm height with 50mm collar height, detachable perforated base with spacer disc of 148mm dia and 47.7mm thick is used to obtain a specimen of exactly 127.3mm height.

Loading Machine

Compression machine operated at a constant rate of 1.25mm/min. Loading frame with cylindrical plunger 50mm dia & dial gauge for measuring the deformation due to application of load.

Compaction rammer

Type of compaction	No of layers	Wt of hammer (kg)	Fall (cm)	No of blows
Light compaction	3	2.6	31	56
Heavy compaction	5	4.89	45	56

Annular weight or surcharge weight

2.5 Kgs of surcharge wt of 147mm dia are placed on specimen both at the soaking and testing of prepared samples.

Procedure

CBR test may be performed on undisturbed soil specimens.

- About 5kgs of soil is taken passing though 20mm IS sieve and retained on 4.75mm IS sieve and the soil is mixed with water up to OMC.
- The spacer disc is placed at the bottom of the mould over the base plate & a coarse filter paper is placed over the spacer disc.
- Then the moist soil sample is to be compacted over this in the mould by adopting either IS light compaction or IS heavy compaction.
- For IS heavy compaction 3 equal layers of compacted thickness about 44mm by applying 56 evenly distributed blows from 2.6kgs rammer.
- For IS heavy compaction 5 equal layers of compacted thickness about 26.5mm by applying 56 evenly distributed blows from 4.89 kg rammer.
- After compacting the last layer, the collar is removed and the excess soil above the top of the mould is evenly trimmed off by means of straight edge (of 5mm thickness).
- Clamps are removed and the mould with compacted soil is lifted leaving below the perforated base plate & the spacer disc which is removed and the mould with compacted soil is weighed
- Filter paper is placed on the perforated base plate & the mould with compacted soil is inverted & placed in position over the base plate and now the clamps of the base is tightened
- Another filter paper is placed on the placed on the top surface of the sample & the perforated plate with adjustable stem is placed over it.
- Now surcharge weights of 2.5 or 5kgs are placed over the perforated plate & the whole mould with the weights is placed in a water tank for soaking such that water can enter the specimen both from the top & bottom.
- The initial dial gauge readings are recorded & the test set up is kept undisturbed in the water tank to allow soaking of the soil specimen for full 4 days or 96 hrs.
- The final dial gauge reading is noted to measure the expansion & swelling of the specimen due to soaking.
- The swell measurement assembly is removed the mould is taken out of the water tank

& the sample is allowed to drain in a perpendicular position for 15 min surcharge wt, perforated plate with stem, filter paper is removed.

- The mould with the soil subgrade is removed from the base plate & is weighed again to determine the wt of water absorbed.
- Then the specimen is clamped over base plate surcharge wt.'s is placed on specimens centrally such that the penetration test could be conducted. The mould with base plate is placed under the penetration plunger of loading machine.
- The penetration plunger is seated +at the centre of the specimen & is brought in contact with the top surface of the soil sample by applying a seating load of 4kgs.
- The dial gauge for measuring the penetration values of the plunger is fitted in position
- The dial gauge of proving ring & the penetration dial gauge are set to 0.
- The load is applied though the penetration plunger at a uniform rate of 1.5mm/min
- The load reading are recorded at penetration reading 0, 0.5, 1.0, 1.5, 2, 2.5, 3, 4, 5, 7.5, 10 & 12.5mm.
- In case the load reading starts decreasing before 12.5mm penetration, the max load & the corresponding penetration values are recorded.
- After the final reading the load is released & the mould from loading machine.
- The proving ring calibration factor is noted so that load dial gauge value can be converted into the load in kg.

Calculation

Swelling or expansion ratio is calculated from the formula.

Expansion ratio = (100 (df – di))/h

Where,

df = Final dial gauge after soaking in mm

di = Initial dial gauge before soaking in mm

h = initial ht of the specimen in mm

CBR = [illegible]

Standard Load values on crushed stone aggregates for specified penetration values

Penetration, mm	Standard Load, kg	Unit Standard Load, kg/cm^2
2.5	1370	70
5.0	2055	105

- Generally, CBR value @ 2.5mm penetration is higher & this value is adopted.
- The initial upward concavity of the load penetration is due to the piston surface not being fully in contact with top of the specimen or when the top layer of soaked soil being too soft.

MODULUS OF SUBGRADE REACTION OF SOIL PLATE BEARING TEST

The plate bearing test has been devised to evaluate the supporting power of subgrade or any other pavement layer by using plates of larger diameter. Plate bearing test was originally meant to find the modulus of subgrade reaction in the Westergards's analysis for wheel load stresses in cement concrete pavement. In the plate bearing test a compressive stress is applied to the soil or pavement layer through rigid plates of relatively large size & the deflection is measurement for various stress values. The deflection level is generally liBRCEd to a low value of 1.25mm to 5mm.

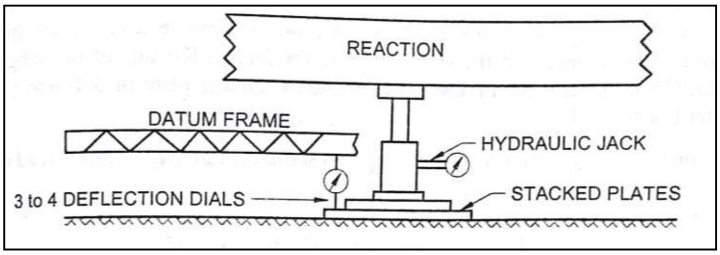


Plate Bearing Test Set Up

Modulus subgrade reaction (k)

- K may be defined as the pressure sustained per unit deformation of subgrade at specified pressure level using specified plate size.
- The standard plate size for finding K value is 75cm dia in same test a smaller plate of 30cm dia is also used (75,60,45,30 & 22.5 cm dia).

Apparatus used Bearing plate

Mild steel of 75cm dia & 1.5 to 2.5 cm thickness.

Loading equipment:

Reaction frame or dead load applied may be measured either by a proving ring or dial gauge assembly.

Settle measurement

It may be made by means of 3 or 4 dial gauge fixed on the periphery of the bearing plate from an independent datum frame. Datum frame should be supported from the loaded area.

Procedure

- At the test site, about 20cm top soil is removed & the site is levelled & the plate is properly seated on the prepared surface.
- The stiffening plates of decreasing dia are placed & the jack & proving ring assembly are fitted to provide reaction against the frame.
- 3 or 4 dial gauges are fixed on the periphery of the palte from the independent datum frame foe measuring settlement.
- A seating load of 0.07 kg/cm2 (320kgs for 75 dia) is applied & released after a few sec.
- The settlement dial gauges reading are now noted corresponding to zero load.
- A load is applied by means of jack sufficient to cause an average settlement of about 0.25mm.
- When there is no perception increase in settlement or when the rate of settlement is less than 0.025mm/min (case of clayey soil or wet soil), the reading of the settlement dial gauge is noted & the avg settlement is found & the load is noted from the proving ring dial reading.
- The load is then increased till settlement increases to a further amount of about 0.25mm & the avg settlement & load are found.
- The procedure is repeated till the settlement reaches 0.175cm.
- A graph is plotted with mean settlement versus mean bearing pressure (load/unit area) as shown in fig.

Bearing pressure settlement curve.

The pressure p (kg/cm2) corresponding to a settlement delta = 0.125cm (obtained from the graph shown above)

The modulus of subgrade reaction k is calculated from the relation given in kg/cm^3

$$K = \frac{p}{\Delta} = \frac{p}{0.125}$$

Correction for smaller plate size

In some cases, the load capacity may not be adequate to cause 75cm dia plate to settle 0.175cm. In such a case a plate of smaller dia (say 30cm) may be used.

Then K value should be found by applying a suitable correction for plate size.

Assuming the subgrade to be an elastic medium with modulus of elasticity E (kg/cm2), the theoretical relationship of deformation (cm) under a rigid plate of radius a (cm) is given by

$$\Delta = \frac{1.18\,(p\,a)}{E}$$

From plate load test we know that, $K = \frac{p}{\Delta}$

Therefore

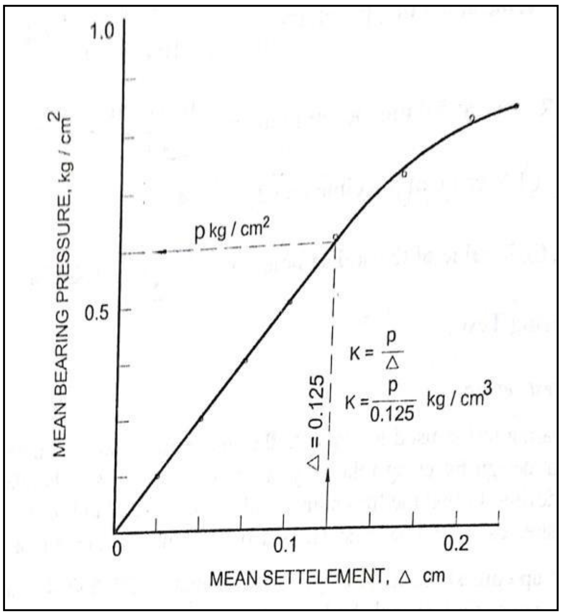
$$K = \frac{p\,E}{1.18\,(p\,a)}$$

$$K = \frac{E}{1.18\,a}$$

If the value of E is taken as constant for a soil, then k x a = constant

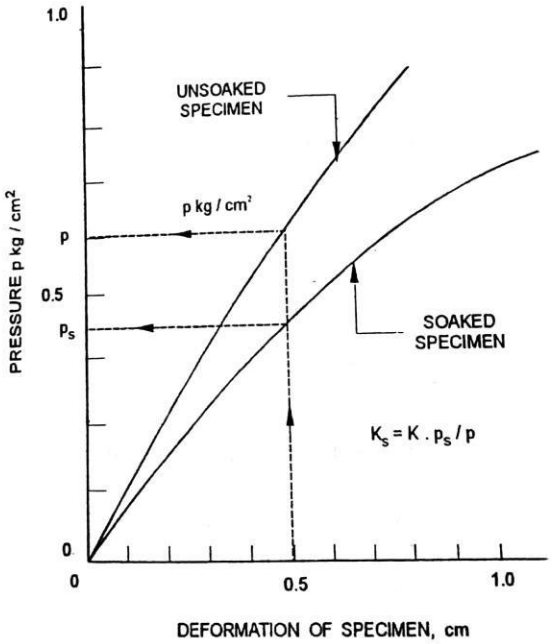
Ie K a = $k_1 a_1$ or $k_{30}\, a_{30} = k_{75}\, a_{75}$

Hence if the test is carried out with a smaller plate of radius a & the modulus of subgrade reaction K is found.



Then the corrected value of modulus of subgrade reaction K for std plate of radius a, is given by

$$\square_{\square\square} = \frac{\square_{\square\square}\ \square\ \square_{\square\square}}{\square_{\square\square}}$$



Allowance for Worst Subgrade Moisture

The modulus of subgrade reaction K of the soil will be lowest at the soaked condition . the moisture content at the time of carrying out plate load test may not represent the worst moisture condition and hence in such cases the value of modulus of subgrade reaction K is found out the prevailing moisture content and the value so obtained is modified by applying a correction factor

$$\square_{\square} = \square_{\square}\frac{\square_{\square}}{\square} = \square_{\square}\left(\frac{\Delta_{\square}}{\Delta_{\square}}\right)$$

Correction for Deflection or Bending of Mild Steel Plate

Given by IS 9214 -1980

$$\square_{\square} = \square.\square\square\square\square\ \square\ \square - \square.\square\square\square\square\ \square\ \square^{\square}$$

AGGREGATES

6.3.1 Functions as Pavement Materials

Stone aggregates form the major portion of pavement structure and they form the prime materials used in the construction of different pavement layers. Aggregates used in various pavement layers have to bear different magnitudes of stresses due to the wheel loads. The aggregates of the pavement surface course have to resist: (i) the wear due to abrasive action of traffic (ii) deterioration due to weathering and (iii) the highest magnitude of wheel load stresses.

The stone aggregates are used in the construction of various pavement layers such as, (i) bituminous pavement layers of flexible pavements (ii) cement concrete mixes used for CC pavement slab and also for other cross drainage structures (iii) granular base course (iv) granular sub-base course or lean cement concrete sub-base and (v) drainage layer. Thus stone aggregates form one of the important components of highway materials and therefore the properties of the aggregates are of considerable significance to the highway engineers.

Most of the road aggregates are prepared by crushing the natural rock. Gravel aggregates are small rounded stones of different sizes which are generally obtained as such from some river beds. Sand due to weathering of rock obtained from river beds is used as fine aggregate. 'Manufactured sand' obtained by crushing of hard rock are also made use of as fine aggregates. The properties of the rock from which the aggregates are formed, depend on the properties of constituent materials and the nature of bond between them. Based on the origin, natural rocks are classified as igneous, sedimentary and metamorphic. Texture is an important factor affecting the property of the rock and the fragments or aggregates.

The aggregates are specified based on their grain size, shape, texture and its gradation. The crushed aggregates of different size are separated by sieving through square sieves of successively decreasing sizes. The required aggregate sizes are chosen to fulfil the desired gradation. The grading, tests and specifications of stone aggregates for different road making purposes have been specified by various agencies like the IRC, BIS, ASTM and BSI.

Hard and soft aggregates

Based on the strength property, the coarse aggregates may be divided as 'hard aggregates' and 'soft aggregates'. Generally for the wearing course of superior pavement types, hard aggregates are preferred to resist the abrading and crushing effects of heavy traffic loads and to resist adverse weather conditions.

Soft aggregates such as moorum, kankar, laterite, brick aggregates and slag have been used in the lower layers of road pavement structure. In the case of low-volume roads soft aggregates and soil-aggregate mixes (as per details given in Chapter – 9) can be advantageously made use of. A different set of tests and specifications are adopted for soft aggregates.

6.3.2 Desirable Properties of Road Aggregates and Tests

Desirable properties

The aggregates used in the pavement layers are subjected to impact due to heavy moving wheel loads. Therefore the aggregates used in pavement layers should have resistance to impact or possess 'toughness' property.

The aggregates used in pavement surface course have to withstand the high magnitude of load stresses and wear and tear. Therefore the aggregates should have sufficient resistance to abrasion caused by traffic movements or should possess 'hardness' property. The aggregates should also have resistance from getting polished or smooth rapidly under traffic movement in order to prevent the pavement surface becoming too slippery particularly under wet surface condition, resulting in accidents due to skidding of high speed vehicles.

The aggregates should have resistance to crushing and be able to retain the strength characteristics during the service life and therefore should possess adequate 'strength'. They should not disintegrate under adverse weather conditions including alternate wet-dry and freeze-thaw cycles or in other words the stones should have enough resistance to weathering action or should possess 'durability' property.

The presence of air voids or pores in stones will result in lower specific gravity and also indicate lower strength characteristics and durability of the stones. The quantum of voids in aggregates is assessed by water absorption test. Higher values of water absorption in coarse aggregates are not desirable for use in bituminous mixes.

The fraction of aggregates which happen to fall in a particular size range, may have varying shapes and as a result may not have the same resistance to crushing and durability when compared with cubical, angular or rounded particles of the same stone. Too flaky and elongated aggregates should be avoided as far as possible as they can get crushed under the roller during compaction and also may break down under heavy wheel loads. Therefore angular shaped coarse aggregates are preferred in flexible pavement layers. The shape factor of coarse aggregates are defined in terms of flakiness index, elongation index and angularity number.

Affinity of aggregates to bituminous binders is an important property of coarse aggregates for use in the bituminous pavement layers. The chemical properties and the surface chemistry of the aggregate particles play important role in performance of bituminous pavements. In case the bituminous mix or the pavement layer is in contact with water for prolonged periods, stripping of bituminous binder is likely to take place from the coated aggregates, if the aggregates do not have affinity to bituminous binder.

The desirable properties of the aggregates may be summarised as follows:

(a) Resistance to impact or toughness.

(b) Resistance to abrasion or hardness.

(c) Resistance from getting polished or smooth/slippery.

(d) Resistance to crushing or crushing strength

(e) Good shape factors to avoid too flaky and elongated particles of coarse aggregates.

(f) Resistance to weathering or durability.

(g) Good adhesion or affinity with bituminous materials in presence of water or less stripping of bitumen coating from the aggregates.

Tests on road aggregates

Tests which are generally carried out for judging the desirable properties and suitability of stone aggregates are listed below:

(a) Aggregate impact test (to assess the toughness or resistance to impact).

(b) Los Angeles abrasion test (to evaluate the hardness and also toughness)

(c) Polished stone value test or accelerated polishing test

(d) Aggregate crushing test (strength characteristics)

(e) Shape tests – flakiness index, elongation index and angularity number

(f) Soundness test or durability test or accelerated weathering test

(g) Specific gravity test and water absorption test.

(h) Bitumen adhesion test or stripping value test of aggregates

All the above mentioned properties of aggregates and tests need not necessarily be conducted; the tests may be decided based on the type of pavement, the pavement layer, importance of the road and location including climatic factors. Some of the important properties and tests that are conducted on road aggregates are given here.

6.3.3 Aggregate Impact Test

During the construction process of pavement layers, particularly compaction by heavy rollers and also due to movement of heavy wheel loads of traffic, the road aggregates are subjected to impact or pounding action and there is possibility of some stones breaking into smaller pieces. The stone aggregates should therefore be sufficiently tough to resist fracture under impact loads. This property could differ from the resistance to crushing of aggregates under gradually increasing compressive stress.

The aggregate impact test is carried out to evaluate the resistance to impact of aggregates to fracture under repeated impacts; the test has been standardised by Bureau of Indian Standards (BIS).

The aggregate impact testing machine consists of a metal base and a cylindrical steel cup of internal diameter 102 mm and depth 50 mm in which the aggregate specimen is placed. A cylindrical metal hammer of weight 13.5 to 14.0 kg having a free fall from a height 380 mm is arranged to drop through vertical guides. The aggregate impact testing machine is shown in Fig. 6.15.

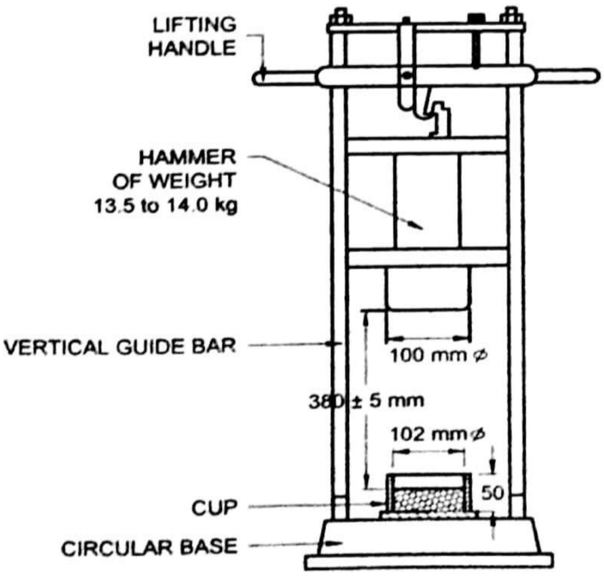


Fig. 6.15 Aggregate impact testing machine

Aggregate specimen passing 12.5 mm sieve and retained on 10 mm sieve is filled in the cylindrical measure in three layers by tamping each layer by 25 blows by the tamping rod. The sample is weighed and transferred from the measure to the cup of the aggregate impact testing machine and compacted by tamping 25 times. The hammer is raised to a height of 380 mm above the upper surface of the aggregate in the cup and is allowed to fall freely on the specimen. After subjecting the test specimen to 15 blows, the crushed aggregate is sieved on 2.36 mm sieve. The aggregate impact value is expressed as the percentage of the fines passing 2.36 mm sieve formed in terms of the total weight of the sample.

The above test is repeated using another specimen of the same aggregate sample, by taking the same weight as in the first test. The mean of the two test results is reported as the Aggregate Impact Value (AIV) of the specimen, to the nearest whole number.

Based on the test results, the toughness property of the aggregate may be reported as given below:

Aggregate impact value, %	Toughness property
Less than 10	Exceptionally tough / strong
10 to 20	Very tough / strong
20 to 30	Good for pavement surface course
Above 35	Weak for pavement surface

The main advantage of aggregate impact test is that test equipment and the test procedure are quite simple; the test can be performed in a short time even at construction site or at stone quarry, as the apparatus is portable. Another advantage is that in addition to measuring the toughness value the test result is considered to give an indirect indication of the strength characteristics.

The aggregate impact value should not normally exceed 30 percent for aggregate to be used in wearing course of pavements. The maximum permissible value is 35% for bituminous macadam and 40% for water bound macadam base courses. The Ministry of Road Transport and Highways (MORTH), Government of India has specified that the AIV of coarse aggregates used in Dense Bituminous Macadam (DBM) binder course and Semi-dense Bituminous Concrete (SDBC) surfacing should not exceed 27 percent and that used in Bituminous Concrete (BC) surface course should not exceed 24 percent.

6.3.4 Los Angeles Abrasion Test

Various abrasion tests on road aggregates

Due to the movements of traffic the road stones used in the surfacing course of pavements are subjected to wearing action at the top surface. Resistance to wear or hardness is hence an essential property for road aggregates, especially when used in wearing course. Thus road stones should be hard enough to resist the abrasion due to the traffic. When fast moving traffic fitted with pneumatic tyres on the wheels move on the road, the soil particles present between the road surface and the tyres cause abrasion of the road surface. Steel tyres of animal drawn vehicles which rub against the stones can cause considerable abrasion of the stones on the road surface. Hence in order to test the suitability of road stones to resist the abrading action due to traffic, different types of abrasion tests are carried out in the laboratory.

Abrasion test on aggregates are generally carried out by any one of the following methods:

(i) Los Angeles abrasion test

(ii) Deval abrasion test

(iii) Dorry abrasion test

Of these tests, the Los Angeles abrasion test is more commonly adopted as the test values of aggregates have been correlated with pavement performance studies. The Bureau of Indian Standards (BIS) has suggested that wherever possible, Los Angeles abrasion test method should be preferred over the Deval abrasion test. While specifying the minimum required resistance to abrasion of coarse aggregates to be used in different types of pavement layers, both the Indian Roads Congress (IRC) and the MORTH have specified Los Angeles abrasion test values only. Therefore in order to test the hardness property or resistance to abrasion of the coarse aggregates, only Los Angeles Abrasion test has been presented in this chapter.

Los Angeles abrasion test

The principle of Los Angeles abrasion test is to find the percentage wear due to the relative rubbing action between the aggregates and steel balls used as abrasive charge. During Los Angeles abrasion test, both abrasion or rubbing action between the aggregates and the steel balls and also impact or pounding action of these balls on the aggregates takes place. Therefore Los Angeles abrasion test is considered to be more reliable for evaluating the suitability of coarse aggregates in pavements as both abrasion and impact occur during the test similar to the field conditions. This test has been standardised by the BIS, ASTM and AASHTO. Acceptable limits of Los Angeles abrasion values of coarse aggregates have been specified by the IRC and also the MORTH.

The Los Angeles machine consists of a hollow cylinder closed at both ends, having inside diameter 700 mm and length 500 mm and mounted so as to rotate about its horizontal axis. A removable steel shelf projecting radially 88 mm into the cylinder and extending to the full length of it is mounted on the interior surface of the cylinder rigidly parallel to the axis. The abrasive charge consisting of cast iron spheres of approximate diameter 48 mm and each of weight 390 to 445 g is placed in the machine. The number of spheres to be used as abrasive charge and their total weight have been specified based on grading of the selected aggregate sample. Los Angeles abrasion testing machine is shown in Fig. 6.16.

The BIS has specified seven sets of grading of coarse aggregates, namely grading A, B, C, D, E, F and G; for each grading different weights of aggregate specimen and abrasive charge have been specified. For grading – A, total 5.0 kg of coarse aggregates consisting of 1250 g each of size ranges, (i) 40 to 25 mm (ii) 25 to 20 mm (iii) 20 to 12.5 mm and (iv) 12.5 to 10 mm are placed in the machine along with abrasive charge consisting of 12 spheres of total weight (5000 g +/– 25 g). For grading – B, total 5.0 kg of coarse aggregates consisting of 2500 g each of the coarse aggregates of size ranges, (i) 20 to 12.5 mm and (ii) 12.5 to 10 mm are placed in the machine along with abrasive charge consisting of 11 spheres of total weight (4584 g +/– 25 g).

The specified weight of aggregate specimen of desired grading is taken, (5 to 10 kg, depending on gradation) and placed in the machine along with the specified abrasive charge. The machine is rotated at a speed of 30 to 33 rpm for the specified number of revolutions (500 to 1000 depending on the grading of the specimen). The abraded aggregate is then sieved on 1.7 mm IS sieve, and the weight of powdered aggregate passing this sieve is found. The result of the abrasion test expressed as the percentage wear or the percentage passing 1.7 mm sieve expressed in terms of the original weight of the sample

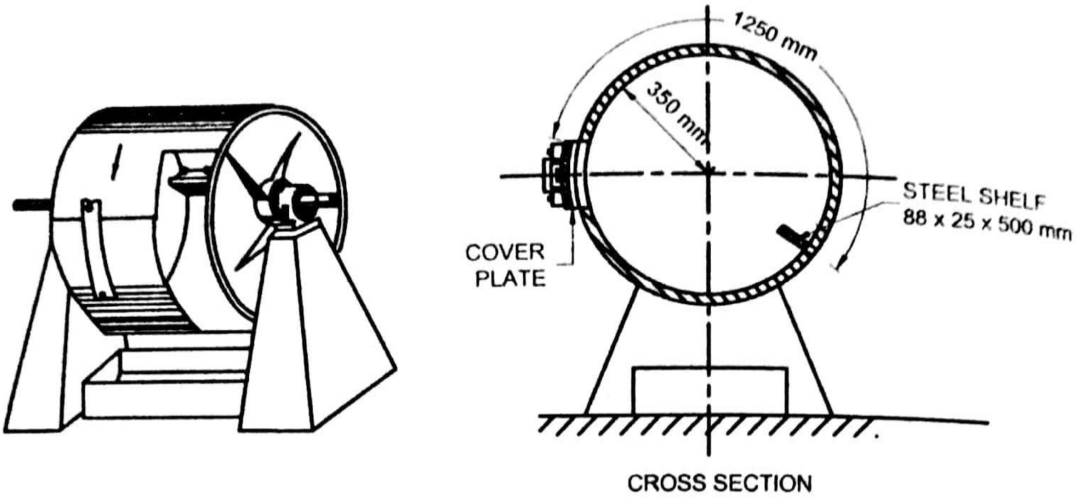


Fig. 6.16 Los angles abrasion testing machine

The Los Angeles abrasion value of good aggregates acceptable for bituminous concrete and other high quality pavement materials should be less than 30 percent; for cement concrete pavement and dense bituminous Macadam (DBM) binder course the maximum acceptable value is 35 percent; values up to 40 percent are allowed in granular base courses (like wet-mix Macadam and water bound Macadam) and in bituminous layers such as bituminous Macadam, bituminous carpet and surface dressing.

6.3.5 Polished Stone Value Test

Importance of the test

The aggregates used in the surface course of pavements are subjected to abrasion and rubbing action due to traffic movements and also during application of brakes. The presence of fine particles of sand and dust between the pavement surface and the tyres of vehicles accelerates the process of the pavement surface getting smoothened along the wheel paths. The smoothened pavement surface becomes slippery under wet conditions, resulting in skidding of high speed vehicles when brakes are applied suddenly and the wheels are locked. Therefore the aggregates used in pavement surface course of high speed highways should have resistance from getting polished or smooth rapidly under traffic movement in order to prevent the pavement surface becoming too slippery resulting in accidents due to skidding of high speed vehicles under wet weather condition.

Principle of the test

The 'Polished Stone Value Test' or the 'Accelerated Polishing Test' on aggregates is conducted in two stages. In the first stage the sample of stone aggregates is placed in a mould and subjected to accelerated polishing action in a machine under standard set of test conditions. In the second stage, the polished sample is subjected to friction test using a pendulum type skid resistance tester to determine the coefficient of friction expressed as a percentage, termed as the 'polished stone value'.

Accelerated polishing

The test specimens are clamped around the rim of the road wheel which can be subjected to accelerated polishing test. The rubber tyred test wheel is lowered until it rests on the surface of the test specimens fixed around the road wheel. The specified weight is added at the end of the lever and the road wheel rotated at a speed of 320 to

325 rpm. Abrading materials and water are released at the specified rate and these are uniformly spread over the surface of the test specimen and the tyre of the test wheel where they are in contact. As the road wheel is rotated, the test specimens are subjected to abrading action or polishing for specified period of time. The machine is stopped and the test specimens are thoroughly cleaned by washing with water to remove sand and other fine particles of stone. The polished set of specimens is now ready for determination of the friction coefficient/skid number or the Polished Stone Value.

Friction coefficient test

The coefficient of friction or the skid resistance value of the test specimen is determined using pendulum type friction tester. The sample of polished specimen is fixed under the sliding portion of the rubber shoe of the pendulum head so as to test the friction coefficient of the sample. The height of the pendulum hinge is adjusted and fixed such that the sliding length of the rubber shoe is 75 mm. The surfaces of the specimen and the rubber shoe are wetted with water.

The pendulum and the pointer are released from the horizontal position and the pointer reading is noted as the 'Skid Number' or the 'Polished Stone Value' from the graduated scale and is recorded. The mean of the two values of the skid number or coefficient of friction expressed as percentage, is reported as the Polished Stone Value of the stone aggregate, to the nearest whole number.

As per the MORTH Specifications, the Polished Stone Value of coarse aggregates used in Bituminous Concrete and Semi Dense Bituminous Concrete surfacing of roads should be not less than 55.

6.3.6 Aggregate Crushing Value Test

The stone aggregates used for the construction of road pavements should possess satisfactory resistance to crushing under the roller during construction and under the application of heavy wheel loads on the pavement during its service life.

The strength of coarse aggregate may be assessed by aggregate crushing test. The aggregate crushing value provides a relative measure of resistance to crushing under gradually applied compressive load. Aggregates possessing high resistance to crushing or low aggregate crushing value are preferred for use in high quality pavements.

The apparatus for the standard test consists of a steel cylinder 152 mm diameter with a base plate and a plunger, compression testing machines, cylindrical measure of diameter 115 mm and height 180 mm, tamping rod and sieves.

Dry aggregate passing 12.5 mm IS sieve and retained on 10 mm sieve is filled in the cylindrical measure in three equal layers, each layer being ramped 25 times by the tamper. The test sample is weighed (equal to W_1 g) and placed in the test cylinder in three equal layers, tamping each layer 25 times. The plunger is placed on the top of specimen and a load of 40 tonnes is applied at a rate of 4 tonnes per minute by the compression machine. The crushed aggregate is removed and sieved on 2.36 mm IS sieve. The crushed material which passes this sieve is weighed equal to W_2 g. The aggregate crushing value is the percentage of the crushed material passing 2.36 mm sieve in terms of original weight of the specimen.

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

The mean of the crushing value obtained in the two tests is reported as the aggregate crushing value, correct to the first decimal place.

Strong aggregates give low aggregate crushing value. The aggregate crushing value for good quality aggregate to be used in base course shall not exceed 45 percent and the value for surface course shall be less than 30 percent. The IRC and BIS have specified that the aggregate crushing value of the coarse aggregates to be used for cement concrete pavement surface should not exceed 30 percent. However aggregate crushing values have not been specified by the IRC or the Ministry of Road Transport and Highways for coarse aggregates to be used in flexible pavement/bituminous pavement construction methods.

6.3.7 Shape Tests

Importance of shape of coarse aggregates

The shape of aggregate particles is determined by the percentage of flaky and elongated particles contained in it. In the case of gravel, the shape may be expressed in terms of the angularity number. Presence of flaky and elongated particles in the coarse aggregates used for the construction of base and surface courses of road pavements is considered undesirable, as these may cause inherent weakness with possibilities of breaking down during compaction as well as under heavy traffic loads. Angular shapes of particles are desirable for granular base course and also for use in bituminous mixes due to increased stability derived from the better interlocking. When the shape of aggregates deviates more from the spherical shape, as in the case of angular aggregates, the void content in an aggregate of any specified size increases and hence the grain size distribution of graded aggregate has to be suitably altered in order to obtain minimum voids or the highest dry density in the dry mix.

The evaluation of shape of the particles is made in terms of flakiness index, elongation index and angularity number.

Flakiness index

Flakiness index (FI) of aggregate is the percentage by weight of aggregate particles the least dimension/thickness of which is less than three fifths or 0.6 of their mean dimension. This test is applicable to sizes larger than 6.3 mm. Standard thickness gauge is used to gauge the thickness or least dimension of the aggregate samples (see Fig. 6.17). The flaky aggregates are those which pass through the designated slots of the thickness gauge which has elongated slots with least dimension equal to 0.6 times of the mean dimension of each size range; these flaky aggregates are separated.

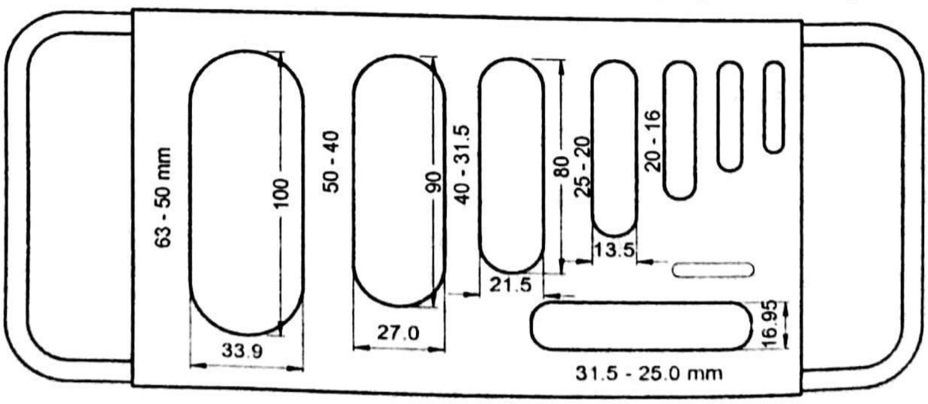


Fig. 6.17 Thickness gauge

The sample of aggregates to be tested is first sieved through a set of sieves and separated into specified size ranges. Now to separate the flaky material, the aggregates which pass through the appropriate elongated slot of the thickness gauge are found. The width of the appropriate slot would be 0.6 of the average of the size range. If the selected size range of aggregate in a group is 20 - 16 mm (i.e., passing 20 mm and retained on 16 mm sieve), the width of the slot to be selected in thickness gauge would be $18 \times 0.6 = 10.8$ mm. The flaky material passing the appropriate slot from each size range of aggregates are added up and let this total weight of flaky particles be W_1 g. If the total weight of sample taken from the different size ranges is W g, the flakiness index is given by $(100W_1)/W$ percent; in other words FI is the percentage of flaky materials, the widths of which are less than 0.6 of the mean dimensions.

The IRC has suggested that the FI of aggregates used in bituminous concrete and surface dressing should not exceed 25 %; the aggregates used in water bound Macadam and bituminous Macadam should not exceed 15 %.

Elongation index

Elongation index (EI) of an aggregate is the percentage by weight of particles, the greatest dimension of which or its length is greater than one and four fifth or 1.8 times their mean dimension. The elongation index test is not applicable for sizes smaller than 6.3 mm. Standard length gauge is used to gauge the greatest dimension or length of the aggregate samples (see Fig. 6.18). The elongated aggregates are those which do not pass through the designated slots of the length gauge which are 1.8 times of the respective mean size of the aggregate; these elongated pieces of aggregates are separated.

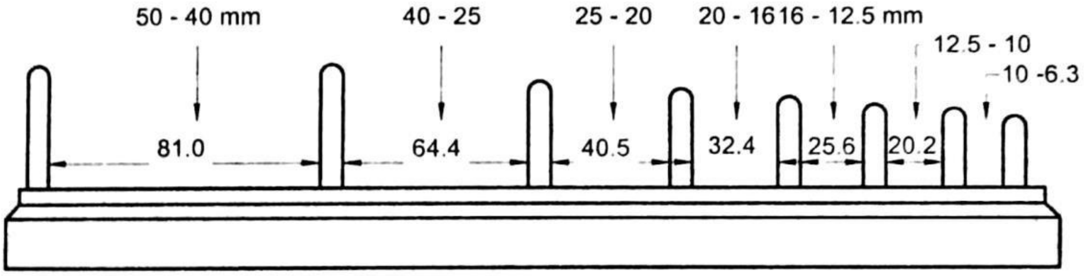


Fig. 6.18 Length gauge

The sample of aggregate to be tested is sieved through a set of sieves and separated into specified size ranges. The longest side of aggregate particles from each of the size range is then individually passed through the appropriate gauge of the length gauge; the gauge length would be 1.8 times the mean size of the aggregate. The portion of the elongated aggregate having length greater than the specified gauge from each size range is weighed. The total weight of the elongated stones is expressed as a percentage of the total weight of the sample taken to obtain the elongation index.

Elongated and flaky aggregates are less workable; they are also likely to break under smaller loads than the aggregate which are spherical or cubical. Flakiness index and elongation index values in excess of 15 percent are generally considered undesirable; however no recognised limits have been laid down for elongation index.

Determination of combined flakiness and elongation index

The Ministry of Road Transport and Highways (MORTH), Government of India has specified the permissible limit of the combined flakiness and elongation index or combined index (CI) for coarse aggregates to be used in different types of pavement layers. MORTH has suggested that the flakiness index test should be carried out on the selected sample of coarse aggregates, as per the Bureau of Indian Standards (BIS) given in the above paragraphs and the value of the flakiness index, FI is determined.

The flaky particles passing through the respective slots of the thickness gauge are removed and the elongation index test is carried out on the remaining non-flaky particles. Let the value of elongation index so determined be EI'. The combined index, CI of the coarse aggregate sample is then equal to (FI + EI').

MORTH has specified the maximum permissible value of the combined index of coarse aggregates as 30 % for wet mix Macadam base course, dense bituminous Macadam binder course and bituminous concrete surface course.

Angularity number

Based on the shape, the aggregate particles may be classified as rounded, partly rounded, irregular, angular or flaky. Angular particles possess well defined edges formed at the intersection of roughly plane faces and are commonly found in aggregates prepared by crushing of rocks. Since weaker aggregates may be crushed during compaction, the angularity number does not apply to any aggregate which breaks down during this test. Angularity or absence of rounding of the particles of an aggregate is a property which is of importance because it affects the interlocking property of compacted aggregate layer and also the ease of handling a mixture of aggregate and binder. The determination of angularity number of an aggregate is essentially a laboratory method intended for comparing the properties of different aggregates for mix design purposes.

The degree of packing of particles of single sized aggregates depends on the shape and angularity of the aggregate. A well compacted single sized rounded aggregates is found to have a solid volume of 67 percent and void volume of 33 percent. The angularity number of aggregate expressed in terms of the voids in a sample of single sized aggregates compacted in a particular manner. Angularity number is defined as (67 – percent solid volume of aggregates). The solid volume of the aggregate is found by filling it in a vessel in a specified manner. Thus, the angularity number measures the voids in excess of 33 percent. The higher the angularity number, more angular is the aggregate. The angularity number for aggregates used in constructions generally range from 0 to 11.

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

The angularity number, AN is found from the formula:

$$AN = 67 - \frac{100W}{CGa} \quad \text{(Eq. 6.10)}$$

This value is expressed as the nearest whole number.

6.3.8 Other Tests on Coarse Aggregates

There are a few other tests that are carried out for assessing the properties of coarse aggregates and their suitability for use in the construction of pavement layers. Of these the basic principle of the following tests are given in this chapter.

(a) Specific gravity and water absorption test

(b) Soundness test

(c) Stripping value test

Specific gravity and water absorption tests

Significance of the tests

The specific gravity of a stone aggregate is considered to be a measure of strength or quality of the material. Stones having low specific gravity are generally weaker than those with higher specific gravity values. The specific gravity tests helps in identification of stone. The specific gravity value of aggregates is made use of for making weight-volume conversions and for calculating the void content in compacted bituminous mixes.

Water absorption is an indicator for the strength of rock. Stones having high water absorption are more porous in nature and are generally considered unsuitable unless they are found to be acceptable based on strength, impact and hardness tests.

Determination of specific gravity and water absorption

About two kg of dry sample of coarse aggregate is placed in wire basket and immersed in water for 24 hours. The sample is weighed in water and the buoyant weight is found. The aggregates are then taken out, surface dried well with absorbent cloth and weighed. The aggregates are then dried in an oven at a temperature 110°C for 24 hours and then the oven dry weight is determined. The specific gravity is calculated by dividing the dry weight of aggregate by weight of equal volume of water. The water absorption is expressed as the percent water absorbed in terms of oven dried weight of the aggregates.

The specific gravity value of rocks generally varies from 2.6 to 2.9. Rock specimens having more than 0.6 percent water absorption are considered unsatisfactory unless found acceptable based on strength tests. However slightly higher value of porosity may be acceptable for aggregates used in bituminous pavement construction, if the aggregates are other-wise found suitable.

Soundness test

Soundness test is intended to study the resistance of aggregates to weathering action by conducting 'accelerated weathering test cycles'. In order to quicken the effects of weathering due to alternate wet-dry and/or freeze-thaw cycles in the laboratory, the resistance to disintegration of aggregate is determined by using saturated solution of sodium sulphate or magnesium sulphate, as per the BIS.

Clean, dry aggregate specimen of specified size range is weighed and the number of pieces counted. The aggregate sample is immersed in the saturated solution of sodium sulphate or magnesium sulphate for 16 to 18 hours. Then the specimen is dried in an oven at 105 to 110°C to a constant weight, thus making one cycle of immersion and drying. The number of such cycles is decided by prior agreement and then the specimens are tested. After completing the final cycle, the sample is dried and each fraction of the aggregate is examined visually to see if there is any evidence of excessive splitting, crumbling or disintegration of the grains. Sieve analysis is carried out to note the variation in gradation from the original. The fraction of coarse aggregate in each size range is sieved through specified sieves.

As a general guidance, the average loss in weight of aggregates to be used in pavement construction after 10 cycles should not exceed 12 percent when tested with sodium sulphate and 18 percent when tested with magnesium sulphate. However the IRC has specified the maximum permissible loss in weight after five wet – dry cycles as 12 percent with sodium sulphate and 18 percent for magnesium sulphate for aggregates to be used in the bituminous binder course and surface course of flexible pavements.

Stripping value of road aggregates

Adhesion of bituminous binder with aggregates

Bituminous binders adhere well to all normal types of aggregates provided they are dry and are not dusty. In the absence of water there is practically no adhesion problem in bituminous road construction. The process of coating the aggregates is controlled largely by the viscosity of the binder. When the viscosity of the binder is high, coating of aggregates by the binder is slower.

Two problems are observed due to the presence of water. First problem is that if aggregate is wet and cold, it is normally not possible to coat with a bituminous binder. The water film from the wet aggregates can be removed by heating the aggregates and mixing can be done at higher temperature. Second problem is 'stripping' or detachment of coated binder from the aggregate due to the presence of water. This problem of stripping is experienced when the bituminous pavement layer is subjected to prolonged soaking under water and the problem is more predominant in bituminous mixes which are permeable to water and when certain types of aggregates are used in bituminous construction work.

The stripping is found be more in certain types of aggregates due to the fact that these aggregates have greater affinity towards water than with bituminous binders; the displacement of bituminous coating from the aggregates depends on the physico-chemical forces acting on the system.

Assessment of suitability of aggregates with respect to adhesion

In order to ascertain the suitability of coarse aggregates for bituminous road construction, it is desirable to study the stripping or displacement characteristics of the binder from the coated aggregates by soaking in water. Several laboratory tests have been developed to assess the adhesion characteristics of the bituminous binder to an aggregate in the presence of water. These tests may be classified into six types (i) Static immersion tests (ii) Dynamic immersion tests (iii) Chemical immersion tests (iv) Immersion mechanical tests (v) Immersion trafficking tests and (vi) Coating tests.

The static immersion test is very commonly used as it is easy and simple. The principle of this type of test is immersing aggregates coated with binder in water and estimating the degree of stripping of the bituminous binder. A stripping test method has been developed by the Road Research laboratory (RRL) England. The method of test for assessing the stripping value of coarse aggregates coated with bitumen has been standardised by the BIS and this test has been briefly presented in this chapter.

Stripping value test

200 g of dry and clean stone aggregate passing 20 mm IS sieve and retained on 12.5 mm sieve is heated up to 150°C. The heated aggregate is mixed with five percent by weight of bitumen binder heated to 160°C. The aggregate and binder are mixed thoroughly till they are completely coated and mixture is transferred to a 500 ml beaker and allowed to cool at room temperature for about two hours. Distilled water is then added to immerse the coated aggregates. The beaker is covered and kept in a water-bath maintained at 40°C, taking care that the level of water in the water-bath is at least half the height of the beaker. After 24 hours, the beaker is taken out, cooled at room temperature and the extent of stripping from the individual aggregates is estimated visually.

The stripping value is the ratio of the average uncovered or stripped area observed visually to the total area of aggregates in each test, expressed as a percentage. The mean of three results is reported as stripping value of the tested aggregates and is expressed as the nearest whole number.

The visual assessment of stripping value is more subjective and may lead to poor reproducibility. But still the test is an indicator as how a mixture of aggregates and binder may behave in the presence of water. Thus, an adhesion test such as the simple static immersion test or the stripping test would be suitable to assess whether the binder would adhere to the aggregate when immersed in water.

The IRC has specified the maximum stripping value as 25 percent for aggregates to be used in bituminous construction like surface dressing, bituminous Macadam and bitumen mastic. The maximum stripping value suggested by IRC is 10 % for aggregates used in open graded premix carpet.

Alternate test method

The degree of bitumen adhesion may also be mechanically measured indirectly by measuring the change in a mechanical property of the compacted bituminous mix, such as compressive strength or any other strength test due to soaking under water. The percentage reduction in strength is an indicator of the extent of damage due to immersing the specimen of bituminous mix in the water.

Methods of dealing with problem of stripping

Most stone aggregates surfaces are electrically charged. As an example, silica a common constituent of igneous rocks possesses a weak negative charge and hence these have greater attraction with the polar liquid water than with bituminous binders having little polar activity. These aggregates which are electro-negative have greater affinity with water and are called 'hydrophillic'. Basic aggregates like lime stones have a dislike for water and greater attraction to bitumen, as they have positive surface charge. These aggregates are called 'hydrophobic'. It is important to know the type of charge of aggregates used in bituminous construction.

If the stripping value exceeds the specified value, use of anti-stripping agents may be recommended. Several anti-stripping agents are available, which when used with the bituminous mix could reduce the stripping.

Now bituminous binders are also available that are either cationic or positive and anionic or negative and hence a suitable selection may be made depending on aggregates available. Cationic (+) bitumen may be selected for electronegative aggregate and anionic (–) bitumen for electropositive aggregates.

BITUMINOUS MATERIALS

6.4.1 Types and Characteristics of Bituminous Binders

Bituminous binders used in pavement construction works are (i) bitumen and (ii) tar. Bitumen is a petroleum product obtained by the distillation of petroleum crude. Coal tar is produced from coal as a by product of coke. Both bitumen and tar have similar appearance as both are black in colour. Though both these binders were used for pavement works, they have widely different characteristics. Tar is no longer used for paving applications because of its undesirable characteristics including high temperature susceptibility and harmful effects of its fumes during heating.

Bitumen is hydrocarbon material of either natural or pyrogenous origin found in gaseous, liquid, semisolid or solid form and is completely soluble in carbon disulphide and in carbon tetra chloride. Bitumen is a complex organic material and occurs either naturally or may be obtained artificially during the distillation of petroleum. Bituminous materials are very commonly used in highway construction because of their binding and water proofing properties. The different grades of bitumen used for pavement construction work of roads and airfields are called paving grade bitumen and those used for water proofing of structures and industrial floors etc. are called industrial grade bitumen.

Paving grade bitumen which is obtained from the distillation process of petroleum crude is extensively used in the construction of flexible pavement layers, particularly the surface and binder courses. At normal range of atmospheric temperature, bitumen is in semi-solid state and remains highly viscous and sticky. When the paving grade bitumen is heated, it softens at a rapid rate and attains fluid consistency and the viscosity decreases with further increase in temperature. For the construction of bituminous pavements, the paving grade bitumen is heated to temperatures in the range of 130 to 175 °C or even higher, depending upon the type and grade of bitumen selected and the type of the construction work. Mixing of the bitumen with the aggregates is done in a hot mix plant to obtain 'hot bituminous mix'.

In order to achieve fluid consistency of the bitumen at relatively low temperatures with nominal heating, 'cut-back bitumen' has been developed. Cutback bitumen is prepared by diluting a paving grade bitumen with a volatile solvent such as a light fuel oil or kerosene. The consistency of the cut-back and the rate at which it hardens after application depends on the grade of the bitumen selected and the characteristics and proportion of the light oil/diluent used.

Another entirely different approach of achieving fluid consistency of bitumen for use in road works without the need to heat the binder is the 'bitumen emulsion'. Bitumen emulsion or emulsified bitumen is prepared by dispersing bitumen in the form of fine globules suspended in water with the help of a suitable emulsifier. The properties of bituminous emulsions vary depending upon the properties of the bituminous binder, its proportion with respect to water and the properties of the emulsifier. Appropriate type and grade of bitumen emulsion may be selected for being directly sprayed as prime coat or tack coat and for being mixed with aggregates to prepare 'cold bituminous mix'.

The viscosity of ordinary paving grade bitumen varies considerably with temperature, resulting in bituminous pavement surface course being susceptible to temperature changes. During hot weather, the bituminous pavement surface course becomes soft and during cold weather it becomes too stiff and brittle with the possibility of early cracking. Bitumen modifiers reduce the temperature susceptibility of the bituminous binder and that of the bituminous mix. Bituminous mixes prepared using suitable type of modified binders offer better resistance to deformation at higher temperatures and remains relatively more flexible and elastic at low temperatures.

Thus the types of bituminous binders that are used in flexible pavement construction are:

(a) Paving grade bitumen

(b) Modified bituminous binders

(c) Cut-back bitumen and

(d) Bitumen emulsion

Of the above binders, the paving grade bitumen and modified bituminous binders need heating before being used in paving applications. Cut-back bitumen may or may not need slight heating depending on the selected grade of the binder and the site temperature during mixing. When bitumen emulsion is used in pavement construction, no heating is required. Bituminous emulsions are also available with modifiers.

6.4.2 Functions of the Binders as Pavement Material and Desirable Properties

Bituminous binders are very commonly used in surface course of pavements; they are also used in the binder and base courses of flexible pavements to withstand relatively adverse conditions of traffic and climate. Bituminous binders are used for preparation of bituminous mixes by mixing with selected aggregates, either in the form of hot bituminous mix or cold mix. Bituminous binders are also used in other techniques of construction such as, 'surface dressing' to be used as a thin surfacing course or in 'penetration Macadam' for use in the base course.

Bituminous binder is used in the form of bitumen emulsion, as a 'prime coat' over granular base course of flexible pavement. The binder in the form of emulsion is also used as a tack coat to be sprayed over the primed base course or over an existing bituminous surface, before laying a bituminous pavement layer.

The bituminous binder (in the form of cut-back or emulsion) may be used in soil – bitumen stabilisation. The bituminous binder may also be used for the preparation of sealer materials for filling the joints and cracks in cement concrete pavements.

The desirable properties of bitumen depend on the type of bituminous construction. In general the bitumen should possess the following desirable properties:

(a) The viscosity of the bitumen at the time of mixing with aggregates and compaction of the pre-mix should be adequate. This is achieved either by (i) heating the bitumen and aggregate prior to mixing or (ii) by using in the form of cut-back or (iii) by using in the form of emulsion of suitable grade

(b) The bituminous binder should become sufficiently viscous on cooling (or on evaporation of the volatile solvent of the cut-back or the water of the emulsion) that the compacted bituminous pavement layer can gain stability and resist deformation under traffic loads

(c) It is desirable that the bitumen binder used in the bituminous mixes form ductile thin films around the aggregates to serve as a satisfactory binder in improving the physical interlocking of the aggregates. The binder material which does not possess sufficient ductility would crack and thus provide pervious pavement surface.

(d) The bituminous binder used should not be highly temperature susceptible. During the hottest weather of the region the bituminous surface should not become too soft or unstable; during cold weather the mix should not become too hard and brittle, causing cracking of surface. The material should also be durable to sustain adverse weathering effects

(e) The bitumen binder should have sufficient adhesion with the aggregates in the mix in presence of water

(f) There has to be adequate affinity and adhesion between the bitumen and aggregate used in the mix. The coated binder should not strip off from the stone aggregate under stagnant water.

6.4.3 Tests on Bitumen

Objects

Bitumen is available in a variety of types and grades. To judge the suitability of these binders various physical tests have been specified by agencies like the Bureau of Indian Standards (BIS), American Society for Testing and Materials (ASTM), Asphalt Institute and the British Standards Institution.

The common tests to assess the properties and requirements of paving grade bitumen are the viscosity tests, penetration test, ductility test and the softening point test. Also specific gravity test and flash and fire point tests are needed for use in paving applications. Additional tests like the matter soluble in carbon-disulphide, loss on heating and penetration test on residue may also be carried out.

Earlier the classification of bitumen was based on the penetration and ductility test results. It was later observed that bitumen from different sources possessing same penetration value at a specified temperature may exhibit entirely different viscosity characteristics and hence different temperature susceptibility characteristics at the application and service temperatures. Therefore it is important to determine the viscosity property of the binder in terms of 'absolute viscosity' and 'kinematic viscosity' test results.

Various tests that are generally carried out to evaluate the properties of bitumen binders are:

(a) Penetration test
(b) Viscosity tests
(c) Ductility test
(d) Softening point test
(e) Specific gravity test
(f) Flash and Fire point tests
(g) Loss on heating test
(h) Solubility test

These importance and principle of these tests are briefly given here.

Penetration test

The consistency of bituminous materials varies depending upon several factors such as constituents, temperature, etc. At temperature ranges between 25 and 50°C most of the paving bitumen grades remain in semi-solid or in plastic state. Determination of absolute viscosity of bituminous materials is not so simple. Therefore the consistency of these materials is determined by indirect methods. Penetration test is one such indirect test to determine the consistency of paving grade bitumen, which is a very simple test.

The penetration test is widely used for classifying the bitumen into different grades. The BIS has standardized the penetration test equipment and the test procedure. The penetration test determines the consistency of these materials for the purpose of grading them by measuring the depth to which a standard needle will penetrate vertically under specified conditions of standard load, duration and temperature. Thus the basic principle of the penetration test is the measurement of the penetration (in units of one tenth of a mm) of a standard needle in a bitumen sample maintained at 25°C during five seconds, the total weight of the needle assembly being 100 g. The concept of the penetration test on bitumen sample is illustrated in Fig. 6.19.

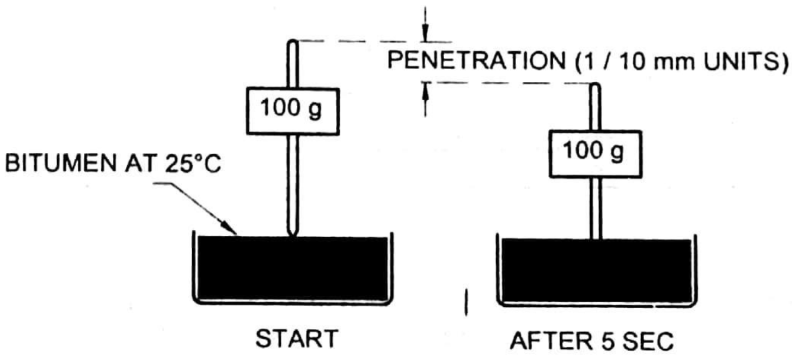


Fig. 6.19 Concept of penetration test on bitumen

Penetration test apparatus or the penetrometer consists of a penetration needle assembly which is attached to a calibrated dial. On release, the penetration needle penetrates into the bitumen specimen without appreciable friction. (See Fig. 6.20) The bitumen is softened to a pouring consistency, stirred thoroughly and poured into containers to a depth at least 15 mm in excess of the expected penetration. The sample containers are then placed in a temperature controlled water bath at a temperature of 25°C for one hour. The sample with container is taken out, placed under the penetrometer and the needle is adjusted to make contact with the surface of the sample. The dial is set to zero or the initial reading is taken and the needle is released for 5 seconds. The final reading is taken on dial gauge.

At least three penetration tests are made on this sample by testing at distances of at least 10 mm apart. After each test, the needle is disengaged and wiped with benzene and dried. The depth of penetration is reported in one-tenth mm units. The mean value of three measurements is reported as a penetration value. It may be noted that the penetration value is largely influenced by any inaccuracy as regards pouring temperature, size of needle, weight placed on the needle and the test temperature.

Penetration test is the most commonly adopted to determine the grade of the bitumen in terms of its hardness because of its simplicity. Softer the bitumen, the greater will be the penetration value. 80/100 bitumen denotes that the penetration value of the binder ranges between 80 and 100. The penetration grades of bitumen binders are generally denoted as 80/100, 60/70 or 30/40 grade bitumen.

Some of the limitations of penetration test for grading of bitumen binders are:

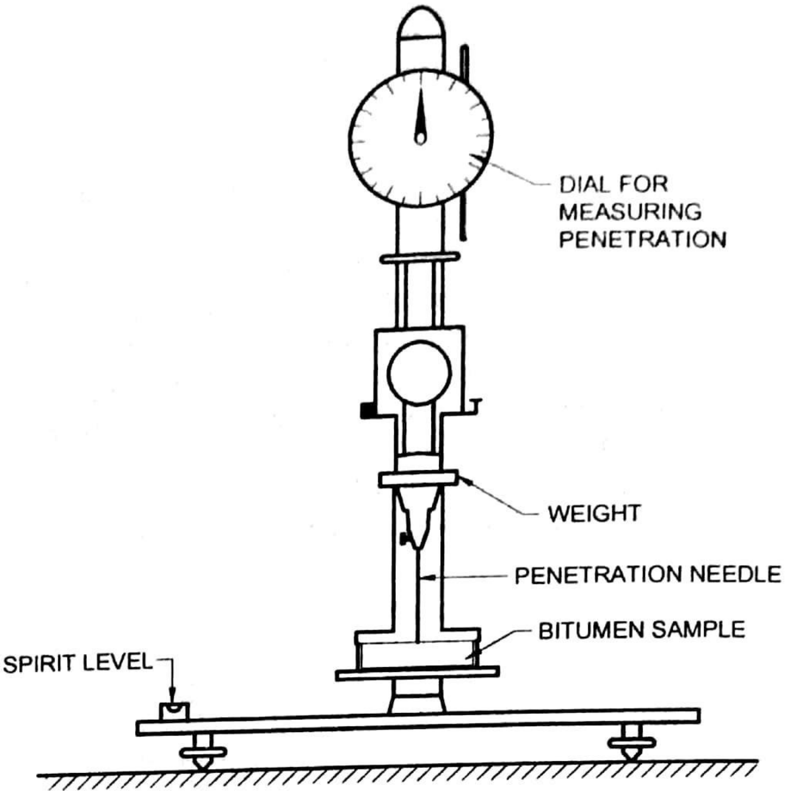


Fig. 6.20 Penetrometer

(i) penetration test is an empirical test and it has no relation with the fundamental properties of the binder (ii) the test temperature of 25°C is not the general pavement service temperature (iii) the service temperature of the pavement is much higher say, about 60°C for most part of the day in several regions (iv) bitumen having the same penetration value may have different performance while in service depending on its temperature susceptibility; this is because bitumen having the same penetration value may have widely varying temperature-stiffness relationship.

In view of the above limitations, grading of bituminous binders is done based on viscosity test results. 'Viscosity Grading' of bitumen has been recommended by the BIS for paving applications.

Viscosity Tests

Viscosity of bituminous binders

Viscosity of a liquid is the property that retards its flow due to internal friction and it is a measure of resistance to flow of the liquid. The flow of a liquid under an applied force will depend on its viscosity; higher the viscosity, slower will be its movement or rate of flow. The range of viscosity of different types of bituminous binders (such as hot bitumen, cutback bitumen or bitumen emulsion) used in road construction vary considerably depending on the type and grade of the binder and the temperature of application. Therefore different test methods are necessary for the determination of the viscosity of the bituminous binders in liquid state and the method chosen will depend upon the viscosity of the binder to be tested and the purpose for which the measurement is required.

A number of test methods and apparatus have been developed for testing of bituminous binders, some of these are empirical methods which give an indirect measure of viscosity, making use of orifice type viscometers and others are for the direct measurement of absolute viscosity. Various terms that are used to express viscosity of bituminous binders are given below.

Absolute viscosity

The ratio between the applied shear stress and the rate of shear is called the coefficient of viscosity or the 'Absolute Viscosity' of the liquid. Absolute or dynamic viscosity (of a Newtonian liquid, in which the shear stress is directly proportional to the rate of shear strain) is the internal friction such that a tangential force of one dyne (or 0.00001 N) acting on planes of unit area separated by unit distance of liquid produces unit tangential velocity. In CGS units the viscosity is measured as gram per cm-second (g/cm-s) or dyne-s/cm^2 and is termed, Poise (P). The SI unit of viscosity is Pascal-second (Pa-s) or Newton – second per square metre (N-s / m^2) and is equal to 10 P.

Kinematic viscosity

Kinematic Viscosity (of a Newtonian liquid) is the ratio of the absolute viscosity to the density of the liquid, both at the same temperature. It is a measure of resistance to flow of a liquid under gravity. The CGS unit used for the measurement of kinematic viscosity is cm^2/second and is called a Stoke (St). In SI units, kinematic viscosity is expressed in units of mm^2/second or in centi-stoke, cSt which is one hundredth of a stoke, ie., 1 mm^2/second = 1 cSt

If kinematic viscosity (in stokes) is multiplied by the specific gravity of bitumen, the absolute viscosity (in poise) can be obtained.

Indirect measurement of viscosity

Viscosity is indirectly measured by determining the time taken by 50 ml of the binder in fluid state to flow through a specified orifice from a cup, under standard test conditions and specified temperature. This method is suitable for measuring viscosity of bitumen emulsion, cut-back bitumen and tar.

Measurement of viscosity of bituminous binders

Some of the important methods of measuring absolute viscosity of bitumen are:

(i) simple shear of a thin film placed between two parallel flat plates, such as the sliding plate viscometer

(ii) shear between rotating coaxial cylinders or cone and cylinder, such coni-cylindrical viscometer or Brookfield viscometer and

(iii) flow through capillary tube, such as vacuum capillary viscometer.

Equipment like sliding plate micro viscometer and Brookfield viscometer are in use for measurement of viscosity of bitumen of all grades irrespective of testing temperature. The viscosity of bitumen can also be measured by a capillary tube viscometer.

In this chapter, the following methods for determination of viscosity of bituminous binders have been briefly presented:

(a) Absolute Viscosity of paving grade bitumen using vacuum capillary tube viscometer

(b) Kinematic Viscosity of bitumen and cutback bitumen using capillary type viscometer

(c) Indirect measurement of viscosity of bituminous emulsion and tar by using orifice viscometers.

Determination of absolute viscosity by vacuum capillary viscometer

A vacuum capillary tube viscometer is generally used to measure the absolute viscosity of bitumen at 60 °C. The viscometer is mounted in a thermostatically controlled water bath or oil bath at uniform test temperature of 60 °C. At this temperature the paving grade bitumen is highly viscous and cannot flow freely through the capillary tube and therefore there is a need to apply vacuum pressure. The time taken (in seconds) for the liquid bitumen to flow through a known distance in a capillary tube is measured and expressed as the viscosity. Depending on the type of fluid, different diameter tubes are to be used and hence the calibration factors supplied by the manufacturer are necessary.

The measured time, t (sec) is multiplied by the calibration factor C of the viscometer in centi-Stokes per second (cSt/sec) to obtain the value of kinematic viscosity in centi-Stokes.

Kinematic viscosity (at test temperature of 135°C / 60°C), cSt = Ct

BIS has standardised three types of 'Vacuum Capillary Viscometers' for the determination of absolute viscosity of bitumen, namely 'Cannon-Manning', 'Asphalt Institute' and 'Modified Coppers' Vacuum Capillary Viscometer'.

Determination of viscosity using orifice viscometer

Viscosity of liquid bituminous binders like bitumen emulsion and tar are determined by indirect method using orifice type viscometers. A specified quantity of the binder (50 ml) is allowed to flow through specified orifice size of the test-cup at a given temperature and the time taken in seconds is recorded as the viscosity value. The test concept is illustrated in Fig. 6.21. As per the specifications of Bureau of Indian Standards, the viscosity values of bitumen emulsions are determined using 'Saybolt Furol' orifice viscometer at test temperatures of 25 °C and 50 °C. The viscosity values of tar are determined using orifice viscometer called 'Tar Viscometer' using either 10 mm or 4 mm size orifice.

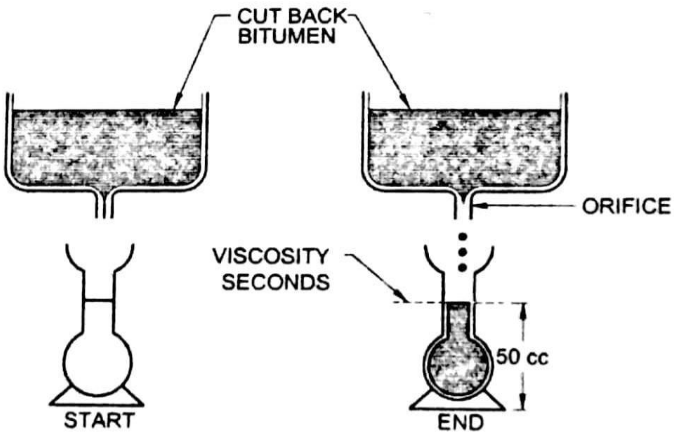


Fig. 6.21 Concept of test using orifice type viscometer

Ductility test

In the flexible pavement constructions where bitumen binders are used, it is important that the binders form ductile thin films around the aggregates. The ductile film of binder improves the physical interlocking of the aggregate-bitumen mixes. Under traffic loads, the bituminous pavement layer is subjected to repeated deformation and recoveries. The binder material which does not possess sufficient ductility would crack and permit the surface water to enter into the pavement resulting in rapid deterioration and failure. Ductility test is carried out on bitumen to test the adhesive property of bitumen and its ability to stretch. The bitumen may satisfy the penetration value, but may fail to satisfy the ductility requirements.

The ductility value is expressed as the distance in centimetre (cm) to which the bitumen specimen of standard size can be stretched before the thread breaks. The standard briquette specimen has a minimum cross section 10 mm × 10 mm. The test is conducted at 27°C with a rate of pull of 50 mm per minute, until the stretched specimen breaks. The ductility test concept is shown in Fig. 6.22.

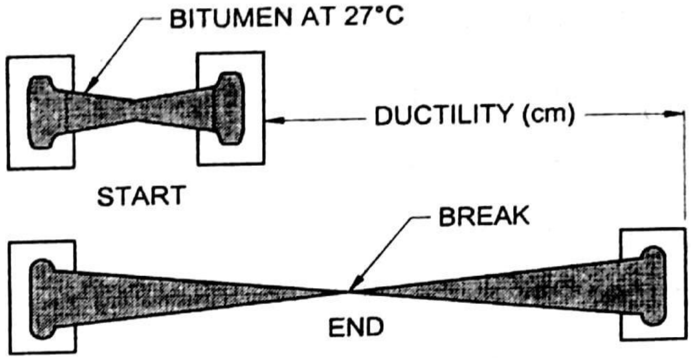


Fig. 6.22 Ductility test concept

The ductility machine functions as a constant temperature water bath with a pulling device at a pre-calibrated rate. Two clips are thus pulled apart horizontally at a uniform speed of 50 mm per minute.

The bitumen sample is heated and poured in the mould assembly placed on a plate. The ductility test specimen and mould are shown in Fig. 6.23. The samples along with the moulds are cooled in air and then in water bath maintained at 27°C. The excess bitumen material is trimmed and the surface is levelled using a hot knife. The mould assembly containing sample is replaced in water bath of the ductility testing machine for 85 to 95 minute. The sides of the mould are removed, the clips hooked on to the machine and the pointer is adjusted to zero. The distance up to the point of breaking of thread is reported as ductility value, in cm. The ductility value gets seriously affected by factors such as pouring temperature, dimensions of briquette, level of briquette in the water bath, presence of air pockets in the specimen briquettes, test temperature and rate of pulling.

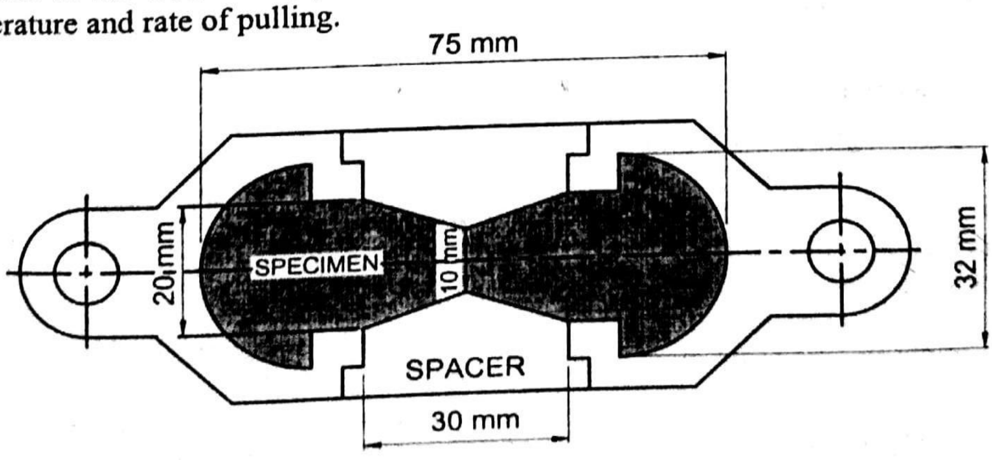


Fig. 6.23 Ductility test specimen and mould

The ductility values of bitumen generally vary from 5 to over 100 for different bitumen grades. A minimum ductility value of 50 to 75 cm is generally specified for bitumen used in pavement construction.

Ductility values have also been specified on residue obtained after conducting 'thin film oven test' (TFOT) on bitumen binder. The BIS has specified that the ductility values on residue from TFOT of paving bitumen of viscosity grades, VG-10, VG-20, VG-30 and VG-40 should not be less than 75, 50, 40 and 25 cm respectively.

Softening point test

The softening point is the temperature at which the substance attains a particular degree of softening under specified condition of test. The softening point of bitumen is usually determined by Ring and Ball test. The concept of softening point test and the test set-up is shown in Fig. 6.24. Generally higher softening point indicates lower temperature susceptibility and is preferred in warm climates.

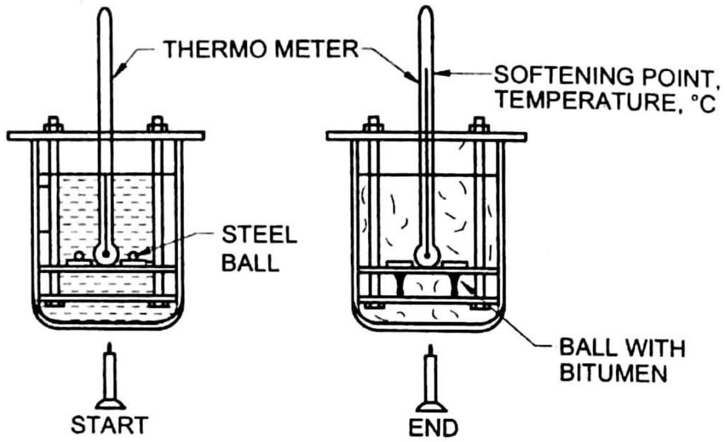


Fig. 6.24 Softening point test set-up

A brass ring containing test sample of bitumen is suspended in a beaker with liquid bath; water is used as the bath if the softening point is less than 80°C and glycerine is used for temperatures exceeding 80°C. A steel ball is placed upon the bitumen sample and the liquid medium is then heated at a rate of 5°C per minute. The temperature at which the softened bitumen touches the metal plate placed at a specified distance below the ring is recorded as the softening point of the bitumen. Harder grades of bitumen possess higher softening point than soft grade bitumen.

The softening point of various bitumen grades used in paving jobs vary between 35° to 70°C.

Specific gravity test

The specific gravity of a bitumen binder is a fundamental property frequently used as an aid to classify the binders for use in paving jobs. In most applications, the bitumen is weighed, but finally when used with aggregate system, the bitumen content is converted on volume basis using density values. The specific gravity value of bitumen is also useful in bituminous mix design. The density of bitumen is influenced by its chemical composition. Increased amounts of aromatic type compounds or mineral impurities cause an increase in specific gravity.

The specific gravity of bituminous binder is defined as the ratio of the mass of a given volume of the binder to the mass of an equal volume of water, the temperature of both being at 27°C. The specific gravity is determined either by using a pyknometer or by preparing a specimen of cube shape in semi-solid or solid state and by weighing in air and water. The specific gravity is obtained by dividing the weight of the bitumen by weight of equal volume of water.

Generally the specific gravity of pure bitumen is in the range of 0.97 to 1.02. The specific gravity of cutback bitumen may be lower depending on the type and proportion of diluent used. Tars have specific gravity ranging from 1.10 to 1.25.

Flash and fire point tests

When a bituminous binder is heated continuously, above a certain temperature it starts emitting volatile vapours and these volatile vapours can momentarily catch fire causing a flash, though the binder itself does not catch fire and burn at this temperature. The temperature at which such behaviour occurs is found to differ for different types and grades of bituminous binders. This condition is very hazardous and therefore it is essential to determine the temperature at which the flash of fire can occur in each type and grade of bituminous binder.

Flash point test gives an indication of the critical temperature at and above which suitable precautions should be taken while heating the binder. In order to eliminate fire hazards during heating, mixing or application, the paving engineers should restrict the mixing and application temperatures well below this temperature.

The 'flash point' of a bituminous binder is defined as the lowest temperature at which application of a test flame causes the vapours of the binder to catch an instant fire in the form of flash under specified test conditions. Flash point test concept is illustrated in Fig. 6.25. Two types of test apparatus may be used for conducting flash point test on bitumen, namely the Pensky-Martens Closed Cup Tester and Open Cup Tester.

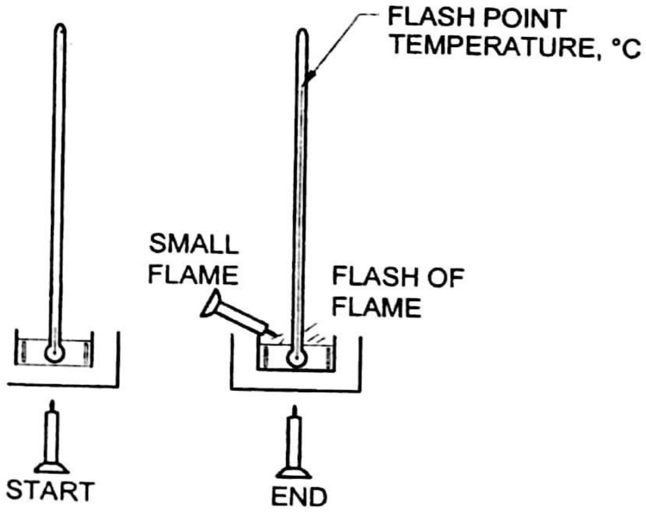


Fig. 6.25 Flash point test concept

If the bituminous binder is further heated to a temperature higher than the flash point, the binder material itself catches fire and continues to burn, the lowest temperature causing this condition is called the 'fire point'. The fire point is always higher than the flash point of a material. The fire point is defined as the lowest temperature at which application of a test flame causes the binder material to ignite and burn at least for five seconds under specified test conditions. Pensky-Martens Open Cup Tester is made use of to determine the fire point of the bituminous binders.

Pensky-Martens closed cup apparatus or open cup are used for conducting the tests. The material to be tested is filled in the cup up to a filling mark. The lid is placed to close the cup in a closed system. All accessories including thermometer of the specified range are suitably fixed. The bitumen sample is then heated at the rate of 5° to 6°C per minute, stirring the specimen. The test flame is applied at intervals depending upon the expected flash and fire points. First application is made at least 17°C below the actual flash point and then at every 1° to 3°C.

The flash point is taken as the temperature read on the thermometer at the time of the flame application that causes a bright flash in the interior of the cup in closed system. For open cup it is the instance when flash appear first at any point on the surface of the material.

In order to determine the fire point, the heating is continued until the material gets ignited and continues to burn for 5 seconds; this temperature is recorded as the fire point.

The BIS has specified the minimum value of flash point by open cup test as 220°C for all the grades of paving bitumen. The minimum specified value of flash point by Pensky-Martens closed cup test for rapid curing cutback bitumen (RC) is 26°C, for medium curing cutbacks, MC 30 and MC 70 is 38°C, for MC 250, MC 800 and MC 3000 is 65°C and for slow curing cutbacks (SC) the minimum specified flash point values range from 65 to 107°C.

Loss on heating test

When bitumen is heated, it loses the volatiles and gets hardened. To study the effect of heating, an accelerated heating procedure is adopted. About 50 g of the sample is weighed and heated to a temperature of 163°C for 5 hours in a special oven designed for this test. After the heating period, this specimen is cooled and weighed again and the loss in weight is expressed as a percentage by weight of original sample.

Bitumen used in pavement mixes should not indicate more than one percent loss in weight; for bitumen of penetration values 150-200 up to two percent loss in weight is allowed. The residue after heating when subjected to penetration test shows a reduction in penetration value. The reduction in penetration value should be less than 40 percent of the original penetration value of the bitumen.

Solubility test

Pure bitumen is completely soluble in solvents like carbon disulphide and carbon tetrachloride. Hence any impurity in bitumen in the form of inert minerals, carbon, salts etc. could be quantitatively analysed by dissolving the samples of bitumen in any of the two solvents. A sample of about 2.0 g of bitumen is dissolved in about 100 ml of solvent. The solution is filtered and the insoluble material retained is washed, dried and weighed; the insoluble material is expressed as a percentage by weight of original sample. The insoluble material should be preferably less than 1.0 percent. In solubility test with carbon tetrachloride, if black carbonaceous residue is over 0.5 percent, the bitumen is considered to be 'cracked'. The minimum proportion of bitumen soluble in carbon disulphide is specified as 99 percent.

Spot test

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

Water content test

It is desirable that the bitumen contains minimum water content to prevent foaming of the bitumen when it is heated above the boiling point of water. The water content in a bitumen specimen is determined by mixing a known weight of the specimen in a pure petroleum distillate free from water, heating and distilling off the water. The weight of the water condensed and collected is expressed as percentage by weight of the original sample. The maximum water content in bitumen should not exceed 0.2 percent by weight.

6.4.4 Grading of Bitumen

In India until recently bitumen binder for use in pavement construction was classified into various 'penetration grades' such as 80/100, 60/70, 30/40, etc. based on the penetration test values determined at 25°C. Now a more rational method of grading paving bitumen, known as 'Viscosity Grading' (VG) has been adopted by the Bureau of Indian Standards (BIS) for grading of bitumen in India, based on the absolute viscosity values determined at 60°C and kinematic viscosity values determined at 135°C. Generally pavement service temperature is considered to be around 60°C and the laying temperature of hot bituminous mixes to be about 135°C. Therefore viscosity grading system based on viscosity tests conducted on bitumen at these temperatures are considered more reliable than the grading method based on penetration test. However two similar viscosity grades of bitumen from different sources may have different viscosity values after exposing to Thin Film Oven Test (TFOT) and hence may behave differently during and after construction.

The four grades of bitumen currently adopted in India based on viscosity values and their respective penetration values at 25 °C are given in Table 6.7.

Table 6.7 Viscosity grading of bitumen and consistency properties

Sl. no.	Viscosity grading	Absolute viscosity at 60°C , poise (min.)	Kinematic viscosity at 135°C, cSt (min.)	Range of penetration value at 25°C
1	VG 10	800	250	80 – 100
2	VG 20	1600	300	60 – 80
3	VG 30	2400	350	50 – 70
4	VG 40	3200	400	40 – 60

The viscosity grades of bitumen recommended for use in India for paving applications are given in Table 6.8.

Table 6.8 Recommended viscosity grades of bitumen for use in India

Viscosity grade (VG)	General applications
VG-40	Use in high stressed areas like intersections, toll plazas, truck terminals, truck lay-byes in lieu of 30/40 penetration grade
VG-30	Paving applications for most part of India, in lieu of 60/70 penetration grade of bitumen
VG-20	Paving applications in cold climatic conditions of North India and in high altitude regions
VG-10	Spraying applications; paving applications in cold regions in lieu of 80/100 penetration grade

BITUMEN EMULSION

Characteristics of bitumen emulsion

A bitumen emulsion is liquid product in which a substantial amount of bitumen is suspended in a finely divided condition in an aqueous medium and stabilized by means of one or more suitable materials. An emulsion is a two phase system consisting of two immiscible liquids; the one being dispersed as fine globules in the other.

The paving bitumen is broken up into fine globules and kept in water. The average diameter of globules of bitumen is about 2 microns. A small proportion of an emulsifier (half to one percent by weight of emulsion) is used to facilitate formation of dispersion and to keep the globules of dispersed binder in suspension. The function of this emulsifier is to form a protective coating around the globules of binder, resisting the coalescence of the globules. Emulsifiers usually adopted is soaps, surface active agents and colloidal powders.

Two common methods followed for the preparation of emulsion are the colloid mill method and the high-speed mixer method. The manufactured emulsion is stored in air tight drums. Bitumen emulsion shall be homogeneous and it should not show un-dispersed bitumen after thorough mixing within one year from the date of manufacture.

The bitumen emulsion may be of anionic type or cationic type. The choice of the type of emulsion for a particular situation depends on the aggregate type, climatic conditions, and environmental conditions. Five types of bitumen emulsions are prepared, namely: (i) Rapid Setting types, RS-1 and RS-2 (ii) Medium Setting type, MS and (iii) Slow Setting types, SS-1 and SS-2.

Tests on bitumen emulsions

The specified tests on bitumen emulsion are given below:

(a) Viscosity test – to assess ability to be sprayed through jets

(b) Water content – to estimate the actual binder quantity

(c) Settlement test – to evaluate settlement when left standing undisturbed

(d) Demulsibility test – to find the residue after mixing with calcium chloride as specified

(e) Miscibility in water – to assess coagulation due to addition of distilled water

(f) Cement mixing test – to assess stability in presence of fines in aggregates

(g) Coating test – to assess coating of stone aggregates

(h) Sieving test – to measure sedimentation of emulsion during storage

(i) Particle charge – to evaluate the type of charge

Uses and applications in road works

Bitumen emulsions have wide range of applications in road construction and maintenance works. The common examples in the construction of bituminous pavement layers are in the interface treatments as prime coat and tack coat and in various other works such as fog seal, seal coat, surface dressing, bituminous carpet, micro-surfacing, etc. The emulsions are extensively being used in maintenance works of bituminous pavements including the patch repair works, particularly during wet weather condition.

When the bitumen emulsion is applied to the road surface, it breaks down and the binder starts coating the aggregates, though needed binding strength develops slowly as and when the water evaporates. The first sign of break-down of emulsion is shown by the change in colour of the film from chocolate brown of the emulsion to black colour of the binder.

The main advantage of bitumen emulsions are (a) they can be used, without heating for spraying or preparing mixes (b) they are particularly useful for patch repair works and can be used even when the surface is wet.

The rapid setting bitumen emulsions are used in spray applications like tack coat, for surface treatments, surface dressing and penetration Macadam.

The medium setting emulsion may be used in cold bituminous mixes in which the percentage of coarse aggregates are substantially high, with a desirable gradation of zero percent fines passing 75 micron sieve and they are also used for surface dressing and penetration Macadam.

The slow setting emulsions are used for prime coat, slurry seal treatments, recycling works and in soil stabilisation; they are also used with well graded bituminous mixes containing a substantial proportion of fine aggregates passing 2.36 mm sieve and a portion containing fines passing 75 micron sieve.

6.4.6 Cutback Bitumen

Characteristics and uses

Cutback bitumen is obtained by blending bitumen binder with suitable volatile diluents or solvents in the required proportion to reduce its viscosity to the desired range. After the cutback mix is used in construction work, the volatile solvent gets evaporated, the binder starts hardening and develops the binding properties. The rate at which the cutback hardens on the road depends upon the characteristics and quantity of the volatile oil used as the diluents and also on the atmospheric temperature and humidity at the work site.

Cutback bitumen binder of appropriate type and grade is selected for use as tack coat without the need to heat. This binder is particularly preferred for use in sites at sub-zero temperatures and in regions of high altitude. Cutback may also be used for preparing bituminous mixes and for soil-bitumen stabilization.

Types of cutback bitumen

Cutback bitumens are available in three types, namely (i) Rapid curing (RC) (ii) Medium Curing (MC) and (iii) Slow Curing (SC).

This classification is based on the rate of curing or hardening after the application, which depends on the type and proportion of diluents/solvent used. Rapid curing cutback bitumen are classified by BIS, on the basis of initial kinematic viscosity into a four grades with designations RC-70, RC-250, RC-800 and RC-3000, in the increasing order of initial viscosity. RC-70 is rapid curing cutback of low initial viscosity to be sprayed at normal air temperature without heating, whereas the RC-800 and RC-3000 are products of high viscosity which cannot be easily mixed with fine aggregate or soil, at low temperatures.

Medium curing (MC) cutback bitumen is classified on the basis of initial viscosity into five grades: MC-30, MC-70, MC-250, MC-800 and MC-3000 in the increasing order of viscosity. MC-30 may be used as primer. Similarly the Slow Curing (SC) cutbacks are classified into four grades and are designated as: SC-70, SC-250, SC-800 and SC-3000.

Properties and tests

RC, MC and SC types of cutback bitumen of the various grades mentioned above should comply with the requirements with regard to the properties such as viscosity at different test temperatures, flash point, distillation fractions, residue from distillation up to the specified temperature and tests on residue from distillation. The following tests are generally carried out on cutback bitumen.

(i) Kinematic Viscosity

(ii) Flash point test (Penskey Marten's closed type)

(iii) Distillation test (to find fractions of distillate up to 190, 225, 260, 315 and 360ºC).

(iv) Tests on residue from distillation up to 360 ºC:

(v) Viscosity at 60 ^{0}C

(vi) Ductility at 27 ^{0}C

(vii) Matter soluble in Trichloro-ethylene

(viii) Water Content

6.4.7 Modified Bituminous Binders

Objects

The viscosity of ordinary paving grade bitumen varies considerably with temperature; as a result the bituminous pavement surface course also becomes susceptible to temperature changes. During hot weather the bituminous surface course becomes soft resulting in possibility of permanent deformation and early rutting along the wheel paths of heavy vehicles. During cold weather, the bituminous pavement surface course becomes too stiff and brittle with the possibility of early cracking under repeated application of heavy wheel loads.

Bitumen modifiers reduce the temperature susceptibility of the binder as well as that of the bituminous mix with consequent improvement in pavement stability by imparting visco-elastic properties to the mix. This product helps to reduce the permanent deformation or rutting of the bituminous surface course under traffic loads. Modified bituminous binders offer better resistance to deformation at higher temperatures and remains flexible and elastic at low temperatures.

The use of virgin polymers to modify the characteristics of the bituminous binder in bituminous mixtures is an accepted practice in the highway construction industry. Properties inherent in polymer additives enhance the performance characteristics of the bituminous binder and of the compacted bituminous pavement layer.

Characteristics

Some of the materials used as modifiers of the bitumen binder are: (a) Polymers – SBS (Styrene-Butadiene-Styrene), SBR (Styrene-Butadiene Rubber), EVA (Ethylene Vinyl Acetate) and (b) Rubber – Crumb rubber and Natural rubber. The advantages of modified binders are improved resistance to cracking as stress or strain absorbing membrane (SAM) and stress absorbing membrane interface (SAMI). They provide reduced temperature susceptibility and more cohesive and tough binders to improve aggregate retention in high stress seal (HSS) applications.

Polymer modified binders (PMB) may be used with bituminous mixes to: (i) improve resistance to permanent deformation (ii) improve fatigue resistance and (iii) increase durability in bituminous mixes. Both elastomeric types and plastomeric types provide improved resistance to deformation as well as improved durability of open graded mixes.

Bituminous mixes produced with elastomeric PMB will generally have a lower stiffness modulus than those with conventional bitumen, but have a significantly high flexibility. Bituminous mixes with plastomeric PMB types may have the same or even higher stiffness than conventional bitumen, but without a large increase in flexibility. Therefore plastomeric type polymers should not be used where a high degree of flexibility is required during cold weather. Bituminous mixes prepared using crumb rubber (powdered scrap rubber) modified bitumen binders have also been used to provide improved flexibility, resistance to deformation and resistance to reflective cracking of bituminous pavement surface course.

Mixes using modified bituminous binders require special design considerations and care during construction operations.

Classification

As per BIS (vide IS 15462: 2004), polymer and rubber modified bitumen are classified into four types as given below:

(a) Type A - PMB (P) : Plastomeric thermoplastics based

(b) Type B - PMB (E) : Elastomeric thermoplastics based

(c) Type C - NRMB : Natural rubber and SBR latex based

(d) Type D - CRMB : Crumb rubber/treated crumb rubber based

Type A, Type B and Type C are further classified into three grades according to their penetration value. Type D is further classified into three grades according to softening point values. The grades of Type A - PMB (P) are as follows:

(i) PMB (P) 120- Type A - PMB (P) having a penetration value between 90 to 150.

(ii) PMB (P) 70- Type A - PMB (P) having penetration value between 50 to 90.

(iii) PMB (P) 40- Type A - PMB (P) having penetration value between 30 and 50.

Similarly the grades of Type B - PMB (E) are (i) PMB (E) 120, (ii) PMB (E) 70 and (iii) PMB (E) 40. The various grades of Type C - NRMB are (i) NRMB 120, (ii) NRMB 70 and (iii) NRMB 40. The grades of Type D - CRMB are (i) CRMB 50, (ii) CRMB 55 and (iii) CRMB 60. In case of CRMB, the numeral indicates the minimum desirable softening point value, viz., CRMB 50 means CRMB having minimum softening point value of 50 °C.

Tests on modified bituminous binders

Elastic recovery test

The elastic recovery test is intended to assess the degree of bitumen modification by elastomeric additives. This is a simple test conducted in a ductility testing machine to optimise the dosage of polymeric additives in bitumen and also helps in assessing the quality of the modified bitumen in the laboratory. The elastic recovery of the modified bitumen is evaluated by comparing the recovery of a thread of modified bitumen after conditioning for one hour at specified test temperature.

The sample is prepared and conditioned as per the procedure of the ductility test but, in the elastic recovery mould shown in Fig. 6.25. The test specimen is elongated at the specified rate of 50 ± 2.5 mm per minute at the specified temperature to a deformation of 10 cm. As soon as the specimen is elongated to a deformation of 10 cm, the specimen is cut into two halves at the mid-point using scissors. The specimen is kept in the water bath in an undisturbed condition for a period of one hour at the specified temperature. After one hour, the elongated half of the specimen is moved back to the position near the fixed half of the test specimen, so that both the pieces of the specimen just touch each other. The length of the recombined specimen is measured as 'D' cm. The elastic recovery of the tested specimen in percent may be computed as:

$$\text{Elastic recovery, \%} = 100\,(10 - D) / 10 \qquad \text{(Eq. 6.11)}$$

Here, D is the length of the combined specimen, cm

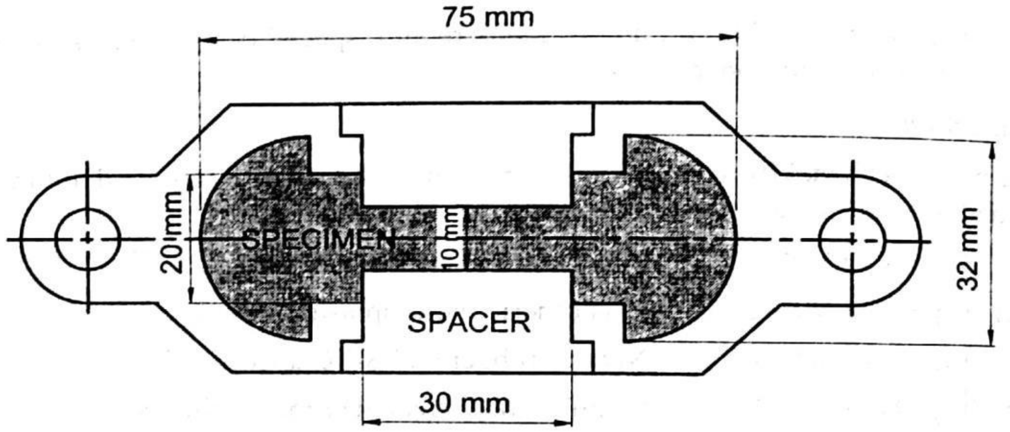


Fig. 6.26 Elastic recovery test specimen and mould

The elastic recovery test can be used to test the suitability of the modified bitumen for paving applications. Modified bitumen with low elastic recovery value is generally found to crack. As per the specifications of the Indian Roads Congress, the requirements of minimum elastic recovery value at 15°C for Elastomeric thermoplastic based polymer modified bitumen is minimum 75% ; for plastomeric polymer modified bitumen is 50%; natural rubber modified bitumen is 30 - 50% and for crumb rubber modified bitumen the elastic recovery should be minimum 50%.

Separation test

In case of modified bitumen there is a possibility that the modifier used in the preparation of the modified bitumen may separate out during storage and transportation. The separation of the modifier such as polymer, crumb rubber, etc., during the hot storage condition is evaluated by comparing the ring and ball softening point values of the samples drawn from top and bottom of conditioned sealed tube of the modified bitumen. The conditioning consist of keeping a sealed tube of modified bitumen in vertical position at 163±5 °C in an oven for a period of 48 hours.

The difference in softening points of the respective top and bottom samples is reported as the separation test value.

Effects of heat and air by thin film oven test

The bituminous binders harden when exposed to atmosphere. The test specimen is subjected to accelerated aging process by 'Thin Film Oven Test' (TFOT) under specified test conditions. The amount of hardening of the bituminous material is evaluated from the reduction in penetration test value, expressed as a percentage of the original penetration value.

Fraass breaking point test

Fraass breaking point is the temperature at which bitumen first becomes brittle as indicated by the appearance of cracks when a thin film of the bitumen on a metal plaque is cooled and flexed in accordance with the specified condition.

Complex modulus test

The complex modulus and phase angle define the resistance to deformation of the binder in the visco-elastic region. The complex modulus and phase angle are used to evaluate performance aspect of modified bitumen, where elastic recovery is insignificant. The test method may be summarised as: (a) preparation of test specimen (b) placement of the specimen in the rheometer maintained at the desired test temperature (c) selection of appropriate strain value and operation using the software and (d) reporting the recorded values of complex modulus (G*) and phase angle (sin ∂).

Choice of modified bituminous binder

Modified bituminous binders are generally recommended for the roads with heavy traffic and the pavement subjected to over-loading conditions. The selection criteria for the type and grade of modified binder are based on atmospheric temperatures at the site of the project road. The softest recommended grade is PMB 120 or CRMB 50, which is used for cold climate areas. PMB 70 or CRMB 55 is used for moderate climate and PMB 40 or CRMB 60 is used for areas with hot climates. For cold climate areas, the properties such as penetration value at 4°C and Fraass breaking point value are to be taken into account.

COMPARISON BETWEEN TAR & BITUMEN

Bitumen	Tar
It has black to dark brown color	It also has black to dark brown in color
It is natural petroleum product	Tar is produced by the destructive distillation of coal or wool
It is soluble in carbon disulphide & in carbon tetrachloride	Tar is soluble only in toluene
It has better weather resisting property	It has inferior weather resisting property
Bitumen are less temp susceptible	Tar is more temp susceptible
Free carbon content is less	Free carbon content is More
It neither binds the aggregate well nor retains the presence of water	It binds aggregate more easily & retain it better in the presence of water.

Introduction to 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.

Requirements of a pavement

The pavement should meet the following requirements:

x Sufficient thickness to distribute the wheel load stresses to a safe value on the sub- grade soil
x Structurally strong to withstand all types of stresses imposed upon it x Adequate coefficient of friction to prevent skidding of vehicles
x Smooth surface to provide comfort to road users even at high speed

Types of pavements

The pavements can be classified based on the structural performance into two, flexible pavements and rigid pavements. In flexible pavements, wheel loads are transferred by grain-to-grain contact of the aggregate through the granular structure. The flexible pavement, having less flexural strength, acts like a flexible sheet (e.g. bituminous road). On the contrary, in rigid pavements, wheel loads are transferred to sub-grade soil by flexural strength of the pavement and the pavement acts like a rigid plate (e.g. cement concrete roads).

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 (see Figure 19:1). The wheel load acting on the pavement will be distributed to a wider area, and the stress decreases with the depth. Taking advantage of this stress distribution characteristic

The lower layers will experience lesser magnitude of stress and less 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).

pavement layer

Typical cross section of a flexible pavement

.

Types of Flexible Pavements

The following types of construction have been used in flexible pavement: x Conventional layered flexible pavement,

x Full - depth asphalt pavement, and

x Contained rock asphalt mat (CRAM).

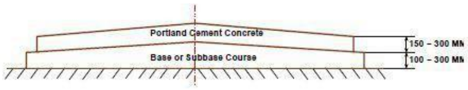
Conventional flexible pavements are layered systems with high quality expensive materials are placed in the top where stresses are high, and low quality cheap materials are placed in lower layers.

Full - depth asphalt pavements are constructed by placing bituminous layers directly on the soil subgrade. 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

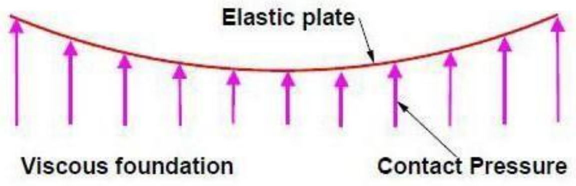
Rigid pavements

Rigid pavements have sufficient flexural strength to transmit the wheel load stresses to a wider area below. A typical cross section of the rigid pavement is shown in Figure 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.



Typical Cross section of Rigid pavement

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,



Elastic plate resting on Viscous foundation

Types of Rigid Pavements

Rigid pavements can be classified into four types: x Jointed plain concrete pavement (JPCP),

x Jointed reinforced concrete pavement (JRCP),

x Continuous reinforced concrete pavement (CRCP), and

x Pre-stressed concrete pavement (PCP).

Jointed Plain Concrete Pavement is plain cement concrete pavements constructed with closely spaced contraction joints. Dowel bars or aggregate interlocks are normally used for load transfer across joints. They normally has a joint spacing of 5 to 10m.

Jointed Reinforced Concrete Pavement Although reinforcements do not improve the structural capacity significantly, they can drastically increase the joint spacing to 10 to 30m. Dowel bars are required for load transfer. Reinforcements help to keep the slab together even after cracks.Continuous Reinforced Concrete Pavement Complete elimination of joints are achieved by reinforcement.

Factors affecting pavement design

Traffic and loading

Traffic is the most important factor in the pavement design. The key factors include contact pressure, wheel load, axle configuration, moving loads, load, and load repetitions.

Contact pressure

The tire pressure is an important factor, as it determines the contact area and the contact pressure between the wheel and the pavement surface. Even though the shape of the contact area is elliptical, for sake of simplicity in analysis, a circular area is often considered.

Wheel load

The next important factor is the wheel load which determines the depth of the pavement required to ensure that the subgrade soil is not failed. Wheel configuration affects the stress distribution and deflection within a pavement. Many commercial vehicles have dual rear wheels which ensure that the contact pressure is within the limits. The normal practice is to convert dual wheel into an equivalent single wheel load so that the analysis is made simpler.

Axle configuration

The load carrying capacity of the commercial vehicle is further enhanced by the introduction of multiple axles.

Moving loads

The damage to the pavement is much higher if the vehicle is moving at creep speed. Many studies show that when the speed is increased from 2 km/hr to 24 km/hr, the stresses and deflection reduced by 40 per cent.

Repetition of Loads

The influence of traffic on pavement not only depends on the magnitude of the wheel load, but also on the frequency of the load applications. Each load application causes some deformation and the total deformation is the summation of all these

Environmental factors

Environmental factors affect the performance of the pavement materials and cause various damages. Environmental factors that affect pavement are of two types, temperature and precipitation.

Equivalent single wheel load

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

x equalancy concept is based on equal stress; x contact area is circular;

x influence angle is 45o; and

x soil medium is elastic, homogeneous, and isotropic half space.

The ESWL is given by:

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

Where P is the wheel load, S is the center to center distance between the two wheels, d is the clear distance between two wheels, and z is the desired depth.

Equivalent single axle load

Vehicles can have many axles which will distribute the load into different axles, and in turn to the pavement through the wheels. A standard truck has two axles, front axle with two wheels and rear axle with four wheels. But to carry large loads multiple axles are provided.

Since the design of flexible pavements is by layered theory, only the wheels on one side needed to be considered. On the other hand, the design of rigid pavement is by plate theory and hence the wheel load on both sides of axle need to be considered. Legal axle load:

Repetition of axle loads:

The deformation of pavement due to a single application of axle load may be small but due to repeated application of load there would be accumulation of unrecovered or permanent deformation which results in failure of pavement.

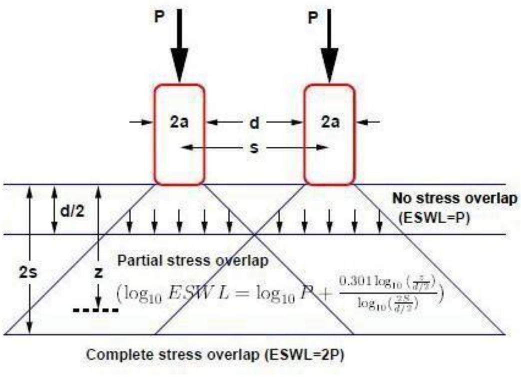
Equivalent axle load factor:

An equivalent axle load factor (EALF) defines the damage per pass to a pavement by the ith type of axle relative to the damage per pass of a standard axle load. While _finding the EALF, the failure criterion is important. Two types of failure criteria‟s are commonly adopted: fatigue cracking and rutting. The fatigue cracking model has the following form:

$$N_f = f_1 \left(\epsilon_t\right)^{-f_2} \times (E)^{-f_3} \, or N_f \propto \epsilon_t{}^{-f_2}$$

Where, Nf is the number of load repetition for a certain percentage of cracking, _t is the tensile strain at the

bottom of the binder course, E is the modulus of elasticity, and f1; f2; f3 are constants. If we consider fatigue



cracking as failure criteria, and a typical value of 4 for f2, then:

$$EALF = \left(\frac{\epsilon_i}{\epsilon_{std}}\right)^4$$

Where, i indicate Ith vehicle, and std indicate the standard axle. Now if we assume that the strain is proportional to the wheel load,

$$EALF = \left(\frac{W_i}{W_{std}}\right)^4$$

Similar results can be obtained if rutting model is used, which is:

$$N_d = f_4 \left(\epsilon_c\right)^{-f_5}$$

where Nd is the permissible design rut depth (say 20mm), s the compressive strain at the top of the subgrade,

and f4; f5 are constants. Once we have the EALF, then we can get the ESAL as given below. Equivalent single axle load, ESAL =

$$\text{Equivalent single axle load, ESAL} = \sum_{i=1}^{m} F_i n_i$$

Where, m is the number of axle load groups, F_i is the EALF for ith axle load group, and ni is the number of passes of ith axle load group during the design period.

Example Let number of load repetition expected by 80 KN standard axle is 1000, 160 KN is 100 and 40 KN is 10000. Find the equivalent axle load. Solution:

Example Solution

i	Axle Load (KN)	No.of Load Repetition (n_i)	EALF (F_i)	$F_i n_i$
1	40	10000	$(40/80)^4 = 0.0625$	625
2	80	1000	$(80/80)^4 = 1$	1000
3	160	100	$(160/80)^4 = 16$	1600

$$\text{ESAL} = \sum F_i n_i = 3225\ kN$$

IRC method of design of flexible pavements

Design traffic

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

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

Initial traffic

Initial traffic is determined in terms of commercial vehicles per day (CVPD). For the structural design of the pavement only commercial vehicles are considered assuming laden weight of three tones or more and their axle loading will be considered. Estimate of the initial daily average traffic flow for any road should normally be based on 7-day 24-hour classified traffic counts (ADT). In case of new roads, traffic estimates can be made on the basis of potential land use and traffic on existing routes in the area.

Traffic growth rate

Traffic growth rates can be estimated

(i) by studying the past trends of traffic growth, and

(ii) By establishing econometric models. If adequate data is not available, it is recommended that an average annual growth rate of 7.5 percent may be adopted.

Design life

For the purpose of the pavement design, the design life is defined in terms of the cumulative number of standard axles that can be carried before strengthening of the pavement is necessary. It is recommended that pavements for arterial roads like NH, SH should be designed for a life of 15 years, EH and urban roads for 20 years and other categories of roads for 10 to 15 years.

Vehicle Damage Factor

The vehicle damage factor (VDF) is a multiplier for converting the number of commercial vehicles of different axle loads and axle configurations to the number of standard axle-load repetitions. It is defined as equivalent number of standard axles per commercial vehicle. The VDF varies with the axle configuration, axle loading, terrain, type of road, and from region to region. The axle load equivalency factors are used to convert different axle load repetitions into equivalent standard axle load repetitions. For these equivalency factors refer IRC: 37 2001. The exact VDF values are arrived after extensive field surveys.

Vehicle distribution

A realistic assessment of distribution of commercial traffic by direction and by lane is necessary as it directly affects the total equivalent standard axle load application used in the design. Until reliable data is available, the following distribution may be assumed.

- **Single lane roads**: Traffic tends to be more channelized on single roads than two lane roads and to allow for this concentration of wheel load repetitions, the design should be based on total number of commercial vehicles in both directions.
- **Two-lane single carriageway roads**: The design should be based on 75 % of the commercial vehicles in both directions.
- **Four-lane single carriageway roads:** The design should be based on 40 % of the total number of commercial vehicles in both directions.

x **Dual carriageway roads**: For the design of dual two-lane carriageway roads should be based on 75 % of the number of commercial vehicles in each direction. For dual three-lane carriageway and dual four-lane carriageway the distribution factor will be 60 % and 45 % respectively.

Numerical example

Design the pavement for construction of a new bypass with the following data:

1. Two lane carriage way
2. Initial traffic in the year of completion of construction = 400 CVPD (sum of both directions)
3. Traffic growth rate = 7.5 %
4. Design life = 15 years
5. Vehicle damage factor based on axle load survey = 2.5 standard axle per commercial vehicle
6. Design CBR of subgrade soil = 4%.

Solution

1. Distribution factor = 0.75
2.

$$\begin{aligned} N &= \frac{365 \times \left[(1+0.075)^{15}-1)\right]}{0.075} \times 400 \times 0.75 \times 2.5 \\ &= 7200000 \\ &= 7.2\ msa \end{aligned}$$

3. Total pavement thickness for CBR 4% and traffic 7.2 msa from IRC:37 2001 chart1 = 660 mm
4. Pavement composition can be obtained by interpolation from Pavement Design Catalogue (IRC:37 2001).
   (a) Bituminous surfacing = 25 mm SDBC + 70 mm DBM
   (b) Road-base = 250 mm WBM
   (c) sub-base = 315 mm granular material of CBR not less than 30 %

Rigid pavement design

Wheel load stresses - Westergaard's stress equation

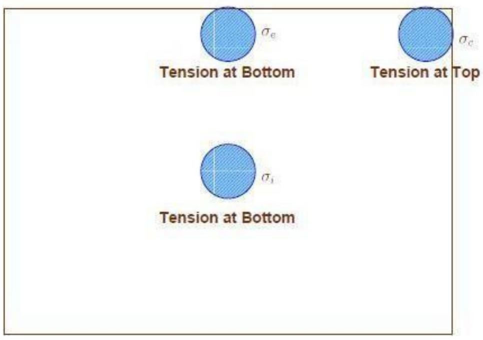
The cement concrete slab is assumed to be homogeneous and to have uniform elastic properties with vertical sub-grade reaction being proportional to the deflection. Westergaard developed relationships for the stress at
interior, edge and corner regions, denoted as _i; _e; _c in kg/cm2 respectively and given by the equation.

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

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

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

where h is the slab thickness in cm, P is the wheel load in kg, a is the radius of the wheel load distribution in cm, l the radius of the relative stiffness in cm 29.1 and b is the radius of the resisting section in cm



Temperature stresses

Temperature stresses are developed in cement concrete pavement due to variation in slab temperature. This is caused by (i) daily variation resulting in a temperature gradient across the thickness of the slab and (ii) seasonal variation resulting in overall change in the slab temperature. The former results in warping stresses and the later in frictional stresses.

Warping stress

The warping stress at the interior, edge and corner regions, denoted as ά ti; ά te; ά tc in kg/cm2 respectively and given by the equation

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

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

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

where E is the modulus of elasticity of concrete in kg/cm^2 (3×10^5), ϵ is the thermal coefficient of concrete per ^{o}C (1×10^7) t is the temperature difference between the top and bottom of the slab, C_x and C_y are the coefficient based on L_x/l in the desired direction and L_y/l right angle to the desired direction, μ is the Poisson's ration (0.15), a is the radius of the contact area and l is the radius of the relative stiffness.

Frictional stresses

The frictional stress ά f in kg/cm2 is given by the equation.

$$\sigma_f = \frac{WLf}{2\times10^4}$$

Where W is the unit weight of concrete in kg/cm2 (2400), f is the coefficient of sub grade friction (1.5) and L is the length of the slab in meters.

Combination of stresses

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

x Summer, mid-day: The critical stress is for edge region given by α critical = α e + α te – αf

x Winter, mid-day: The critical combination of stress is for the edge region given by α critical = α e+ α te + α f

x Mid-nights: The critical combination of stress is for the corner region given by α critical = α c + α tc

MODULE 4: Introduction to 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, favourable light reflecting characteristics, and low noise pollution.

Requirements of a pavement

The pavement should meet the following requirements:

- Sufficient thickness to distribute the wheel load stresses to a safe value on the sub grade soil
- Structurally strong to withstand all types of stresses imposed upon it
- Adequate coefficient of friction to prevent skidding of vehicles
- Smooth surface to provide comfort to road users even at high speed

Types of pavements

The pavements can be classified based on the structural performance into two, flexible pavements and rigid pavements. In flexible pavements, wheel loads are transferred by grain-to-grain contact of the aggregate through the granular structure. The flexible pavement, having less flexural strength, acts like a flexible sheet (e.g. bituminous road). On the contrary, in rigid pavements, wheel loads are transferred to sub-grade soil by flexural strength of the pavement and the pavement acts like a rigid plate (e.g. cement concrete roads).

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 (see Figure 19:1). The wheel load acting on the pavement will be distributed to a wider area, and the stress decreases with the depth. Taking advantage of this stress distribution characteristic the lower layers will experience lesser magnitude of stress and less 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) pavement layer

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 subgrade. 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. A typical cross section of the rigid pavement is shown in Figure 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,

Types of Rigid Pavements

Rigid pavements can be classified into four types:

- Jointed plain concrete pavement (JPCP),
- Jointed reinforced concrete pavement (JRCP),
- Continuous reinforced concrete pavement (CRCP), and
- Pre-stressed concrete pavement (PCP).

Jointed Plain Concrete Pavement is plain cement concrete pavements constructed with closely spaced contraction joints. Dowel bars or aggregate interlocks are normally used for load transfer across joints. They normally has a joint spacing of 5 to 10m.

Jointed Reinforced Concrete Pavement Although reinforcements do not improve the structural capacity significantly, they can drastically increase the joint spacing to 10 to 30m. Dowel bars are required for load transfer. Reinforcements help to keep the slab together even after cracks. Continuous Reinforced Concrete Pavement Complete elimination of joints are achieved by reinforcement.

Factors affecting pavement design

Traffic and loading

Traffic is the most important factor in the pavement design. The key factors include contact pressure, wheel load, axle configuration, moving loads, load, and load repetitions.

Contact pressure

The tire pressure is an important factor, as it determines the contact area and the contact pressure between the wheel and the pavement surface. Even though the shape of the contact area is elliptical, for sake of simplicity in analysis, a circular area is often considered.

Wheel load

The next important factor is the wheel load which determines the depth of the pavement required to ensure that the subgrade soil is not failed. Wheel configuration affects the stress distribution and deflection within a pavement. Many commercial vehicles have dual rear wheels which ensure that the contact pressure is within the limits. The normal practice is to convert dual wheel into an equivalent single wheel load so that the analysis is made simpler.

Axle configuration

The load carrying capacity of the commercial vehicle is further enhanced by the introduction of multiple axles.

Moving loads

The damage to the pavement is much higher if the vehicle is moving at creep speed. Many studies show that when the speed is increased from 2 km/hr to 24 km/hr, the stresses and deflection reduced by 40 per cent.

Repetition of Loads

The influence of traffic on pavement not only depends on the magnitude of the wheel load, but also on the frequency of the load applications. Each load application causes some deformation and the total deformation is the summation of all these.

Environmental factors

Environmental factors affect the performance of the pavement materials and cause various damages. Environmental factors that affect pavement are of two types, temperature and precipitation.

EQUIVALENT SINGLE WHEEL LOAD

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

- Equalancy concept is based on equal stress;
- Contact area is circular;
- Influence angle is 45 degree; and
- Soil medium is elastic, homogeneous, and isotropic half space.

Where P is the wheel load, S is the center to center distance between the two wheels, d is the clear distance between two wheels, and z is the desired depth.

Equivalent single axle load

Vehicles can have many axles which will distribute the load into different axles, and in turn to the pavement through the wheels. A standard truck has two axles, front axle with two wheels and rear axle with four wheels. But to carry large loads multiple axles are provided. Since the design of flexible pavements is by layered theory, only the wheels on one side needed to be considered. On the other hand, the design of rigid pavement is by plate theory and hence the wheel load on both sides of axle need to be considered. Legal axle load.

Repetition of axle loads:

The deformation of pavement due to a single application of axle load may be small but due to repeated application of load there would be accumulation of unrecovered or permanent deformation which results in failure of pavement.

IRC method of design of flexible pavements

Design traffic

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

1. Initial traffic in terms of CVPD

2. Traffic growth rate during the design life

3. Design life in number of years

4. Vehicle damage factor (VDF)

5. Distribution of commercial traffic over the carriage way.

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Initial traffic is determined in terms of commercial vehicles per day (CVPD). For the structural design of the pavement only commercial vehicles are considered assuming laden weight of three tones or more and their axle loading will be considered. Estimate of the initial daily average traffic flow for any road should normally be based on 7-day 24-hour classified traffic counts (ADT). In case of new roads, traffic estimates can be made on the basis of potential land use and traffic on existing routes in the area.

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Rigid pavement design

Wheel load stresses - Westergaard's stress equation

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where h is the slab thickness in cm, P is the wheel load in kg, a is the radius of the wheel load distribution in cm, l the radius of the relative stiffness in cm 29.1 and b is the radius of the resisting section in cm

Temperature stresses

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

(i) Daily variation resulting in a temperature gradient across the thickness of the slab

(ii) Seasonal variation resulting in overall change in the slab temperature. The former results in warping stresses and the later in frictional stresses.

Warping stress

The warping stress at the interior, edge and corner regions, denoted as ά ti; ά te; ά tc in kg/cm^2 respectively and given by the equation

Frictional stresses

The frictional stress ά f in kg/cm2 is given by the equation.

Where W is the unit weight of concrete in kg/cm2 (2400), f is the coefficient of sub grade friction (1.5) and L is the length of the slab in meters.

Combination of stresses

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

Summer, mid-day: The critical stress is for edge region given by α critical = α e + α te – αf

Winter, mid-day: The critical of stress is for the edge region given by α critical = α e+ α te + α f

Mid-nights: The critical of stress is for the corner region given by α critical = α c + α tc

Module 5: HIGHWAY ECONOMICS & FINANCE

INTRODUCTION

Better highway system provides varied benefits to the society. Improvements in highway results in several benefits to the road users such as :

- Reduction in vehicle operational cost per unit length of road.
- saving travel time and resultant benefits in terms of time cost of vehicles and the passengers
- Reduction in accident rates.
- Improved level of service and ease of driving.
- Increased comfort to passengers.

Therefore he level of service of a road system may be assessed from the benefits to the users

The improvement in road net work also benefits the land owner by providing better access and consequently enhancing the land value. The cost of improvements in the highway of land, materials, construction work and for the other facilities should be worked out. From the point of view of economic justification for the improvements, the cost reductions to the highway users and other beneficiaries of the improvements during the estimated period should be higher than the investments made for the improvement. In the planning and design of highways there is increasing need for analysis to indicate justification of the expenditure required and the comparative worth of proposed improvements, particularly when various alternatives are being compared.

The government or any other agency finances highway developments. The funds for these are generally recovered 1ins the road users in the form of direct and indirect taxations. Highway Finance deals with various methods of raising and or providing money for the highway projects.

HIGHWAY USER BENEFITS

General Benefits

Several benefits are brought to highway users and others due to the construction of a new highway or by improving a highway. Road user benefits are the advantages, privileges or savings that accrue to drivers or owners through the use of one highway facility as compared with the use of another. The various benefits due to highway improvement may be classified into two categories: (i) quantifiable or tangible benefits in terms of market values and (ii) non quantifiable or intangible benefits.

Quantifilab1e Benefits

Various benefits which can be quantified include benefits to road user such as reduction in vehicle operation cost, time cost and accident cost. The other benefits include enhancement in land value. These are briefly explained below:

- Saving in vehicle operation cost is due to reduction in fuel and oil consumption and reduction in wear and tear of tyres and other maintenance costs. A road with sharp curves and steep grades require frequent speed changes; presence of intersections require stopping idling and accelerating; vehicle operation on road stretches with high traffic volume or congestion necessitates speed changes and stopping and increased travel time.

Non-quantable Benefits

The non-quantifiable benefits due to improvements in highway facilities include reduction in fatigue and discomfort during travel, increase in comfort and conveniences and improvement in general amenities, social and educational aspects, development of recreational and medical services, improved mobility of essential services and defence forces, aesthetic values, etc..

Motor Vehicle Operation Cost

The factors to be considered for evaluating motor vehicle operation cost would differ depending on the purpose of the analysis. The vehicle may be classified in different groups such as passenger cars, buses, light commercial vehicles, single unit trucks combination vehicles etc., for the purpose of cost analysis. The motor vehicle operation costs depend on several factors which may be grouped .as given below:

- Cost dependent on time expressed as cost per year such as interest on capita depreciation cost, registration fee, insurance charges, garage rent, driver's license salaries etc. as applicable.
- Cost depending on distance driven expressed as cost per vehicle-kilometer. The items which may be included here are fuel, oil, tyres, maintenance and repairs etc.
- Cost dependent on speed include cost of fuel, oil and tyre per vehicle-km-time-cost of vehicles, travel time value of passengers, etc.
- Cost dependent on type of vehicle and its condition. Operation costs of larger vehicles are comparatively higher. The operation cost of old vehicles maintained in poor condition is also higher.
- Accident costs.

The costs of vehicle operation and time for unit distance may be taken as:

$$T = a + (b+c) \qquad (14.1)$$

Speed

Where

a	=	running cost per unit distance, independent of journey time
b	=	a fixed hourly cost, dependent on speeds
c	=	the portion of the running cost which is dependent on speed

Therefore the operation costs may be considered to consist various components like motor fuel cost, lubricating oil consumption, tyre costs, vehicle repair and maintenance, depreciation, cost due to slowing, stopping, idling and standing delays, costs related to pavement surface and its condition, grades, curves and traffic volumes. Also the time costs and accident costs are taken into consideration.

Example 14.1

Calculate the operating cost of a passenger car for 100 km length of a rural highway with no sharp curves for most economical speed of vehicles operation using the following

HIGHWAY COSTS

General

The total Highway Cost for road user benefit analysis is the sum of the capital costs expressed on an annual basis and the annual cost of maintenance. The total cost for highway improvement is obtained from the estimate prepared from the preliminary plans. The total cost of each highway engineering improvement proposal is calculated from the following five components

(i) Right of way

(ii) Grading drainage, minor structures

(iii) Major structures like bridges

(iv) Pavement and appurtenances

(v) Annual cost of maintenance and operation

Computation of total annual highway cost based on summation of the annual cost of individual items of improvements and their average useful lives is considered to be a proper and accurate approach.It is difficult to estimate the service lives of highway elements as there are several variables such as soil, climate topography and traffic. Road life studies enable estimation of lives of pavements, bridges and other roadway facilities.

Annual Highway Cost

The items to be included while computing annual highway cost are

(i) Administration (a portion) Personal service, building, equipment operation, office, insurance etc.

(ii) Highway operation Equipment. building vehicle operation including capital costs of vehicle.

(iii) Highway maintenance

(iv) Highway capital cost : Cost of highway components such as right of way, damage, earthwork, drainage system. pavement bridges and traffic services depreciation cost and interest on investment.

(v) Probable life and salvage value at the end of this period.

The average annual highway cost for a road system may be summed up by the formula.

$$Ca – H + T + M + Cr$$

where

Ca	=	average annual cost of ownership and operation
H	=	average cost for administration and management at head quarters
T	=	average annual highway operation cost.
M	=	average annual highway maintenance cost.
Cr	=	average annual capital cost of depreciation of investment capital or the capital recovery with return on capital

The annual cost is considered in the economic assessment of highway projects. Instead of considering the overall cost of a project the annual repayment of a capital loan plus the interest over a specified period of time of the annual capital cost is considered in the analysis. The first cost of a capital improvement is converted into equivalent uniform annual cost by the formula:

$$Cr = P \left(\frac{i(1 - i)n}{(1+i)n-1} \right)$$

$$= P\ (CRF)$$

Where

Cr = receipt in a uniform series for n periods to cover P at a rate of interest i

P = first cost of improvement of an element of a highway

i = rate of interest per unit period

n = period of time in number of interest periods

$$CRF = \text{Capital recovery factor} = \frac{i(1 - i)n}{(1+i)n-1}$$

At the end of the service life of road pavement, some of the items could be assigned some salvage value. However the salvage value of some other items may be negligible.

The average annual capital cost Cr for a project considering salvage value may be estimated by the use of the formula (for the capital-recovery with salvage value):

$$Cr = (C-Vs) \left(\frac{i(1 - i)n}{(1+i)\ n-1} \right) + I\ Vs$$

$$= (C- Vs)\ CRF + i\ Vs \qquad (14.4)$$

Where C = total investment on construction

Vs = salvage value at the end of n years

i = interest rate applicable

n = number of years of expected use of the facility

The compound amount accumulated sum S on the principal sum of proposed improvement cost or single payment P, including interest rate, i in n years is given by:

$$s = P (1 + i)^n \quad (14.5)$$

economical proposal among various alternatives, in the analysis for economic justification of the proposed improvement, it is required to use judgment such as quantitative selection of the factors in which annual highway cost depends and the estimation of AADT of each class of vehicle considering the normal increase in traffic and the generated traffic.

Methods of Analysis

The procedure for the economic evaluation of highway projects consists of qualification for cost component and the benefits arising out of the project and to evaluate by one of the methods of analysis.

There are several methods of economic analysis. Some of the common methods are. Annual-cost Method, Rate-of-Return Method and Benefit-Cost Method.

Annual-Cost Method

The annual cost of each element of capital improvement is found by multiplying by the appropriate CRF value calculated for the assume life span. The annual cost Cr may be found using the relation (Eq. 14.3).

$$C1 = P. \frac{i(1+i)^n}{(1+i)^n-1} = P(CRF)$$

Rate-of-Return Method

There are number of variations for the determination of raw of return of a highway improvement. In the rate of return method, die interest rate at which two alternative solutions have equal annual cost is found, If the rate of return of all proposed projects are known, the priority for the improvement could be established.

Benefit Cost ratio Method

Principle of this method is to assess the merit of a particular scheme by comparing the annual benefits with the increase in annual cost

Benefit cost ration = Annual benefits from improvement / Annual cost of the improvement

$$= \frac{R - R1}{H1 - H}$$

Where R = total annual road user cost for axisting highway

R1 = total annual road user cost for proposed highway improvement

H = total annual cost of existing road

H1 = total annual cost of proposed highway improvement

The benefit-cost ratios are determined between alternate proposals and those plans dub are not attractive are discarded. Then the benefit cost ratios for various increments of added investment are computed to arrive at the best proposal. hi order to justify the proposed improvement, the ratio should be greater than 1.0. However, the choice of interest rate would affect the results of the benefit-cost solutions.

Total annual road user cost for proposal B = RB = Rs. 2491,125

Benefit-cost ratio,

B = RA-RB = 3081,330 - 2491.125 = 590,205 = 2.874

A HB-HA 381,900 - 176,527 205, 373

Total annual highway cost of proposal C = HC = Rs.3,75,100

Total annual highway cost of proposal C= HC = Rs.2377,245

Benefit – cost ratio,

C = RA-RB = 3081,330 - 2377.245 = 704,085 = 3.546

A HC-HA 375,100 - 176,527 198, 573

Therefore, alternative C is the best one with higher benefit-cost ratio.

HIGHWAY FINANCE

Basic principle in highway financing is that the funds spent on highways are recovered from the road users. The recovery may be both direct and indirect.

Two general methods of highway financing are:

Pay-as-you-go method

Credit financing method

In pay-as-you-go method, the payment for highway improvements, maintenance and operation is made from the central revenue. In credit financing method, the payment for highway improvement is made from borrowed money and this amount and the interests are re-paid from the future income.

Distribution of highway cost

The question of distributing highway cost among the Government, road-user and other has been a disputed task in several countries. Many economists are of the view that the financial responsibility for roads should be assigned only among the beneficiaries on the basis of the benefit each one receives.

There are several theories suggesting the method of distribution of highway taxes between passenger cars and other commercial vehicles like the trucks. However in India the annual revenue from transport has been much higher than the expenditure on road development and maintenance. Therefore there is no problem of distributing the highway cost among other agencies. Also the taxation on vehicles is being considered separately by the states and there seems to be no theory followed for the distribution of taxes between various classes of vehicles.

Sources of Revenue

The various sources from which funds necessary for highway development and maintenance may be made available, are listed below:

Taxes on motor fuel and lubricants.

Duties and taxes on new vehicles and spare part including tyres

Vehicles registration tax.

Special taxes on commercial vehicles

Other road user taxes

Property taxes

Toll taxes

Other funds set apart for highways

There should be an equitable distribution of revenues available for highways.

Highway financing in India

The responsibility of financing different roads lies with the Central Government, State Governments and local bodies including Corporations, Municipalities, District Boards and Panchayats.

Taxes levied by Central Government for highway financing are:

Duties arid taxes on motor fuel

.Excise duty on vehicles and spare parts, tyre etc.

Excise duty on oils, grease, etc

Taxes levied by the State Governments include:

Registration fees for vehicles and road tax

Permits for transport vehicles

Passenger tax on buses

Sales tax on vehicle parts tyre etc.

Fees on driving licenses

Taxes levied by local bodies are mainly the toll tax.

Ever since the introduction of Central Road Fund (CRF) in the year 1929 by taxing motor fuel, this has been the main source of finance for the State Government to meet the road development needs, without having to go through the time consuming process of special sanctions each time. However of late the CRF is also being merged with the general revenue, in March 1976 the Lok Sabha has passed the resolution Of the Ministry of Transport ensuring the existence of the CRF separately with the specified objectives. An Amount of not less than 3.5 paise per litre out of the duty of customs and excise on motor spirit would be set apart towards the CRF for the road development. While utilizing this fund, greater attention would be given to schemes of all-India importance. Twenty percent of the fund would be retained by the central Government as reserve. The fund will also be used for road research schemes, traffic studies, economics surveys and training arrangements for young engineers. The gross revenue from road transport in India during the sixth plan period 1978-83, 1980-85 was about Rupees l2,000 Crores