CIVIL ENGINEERING

For

UPSC Engineering Services Examination, GATE,

State Engineering Service Examination & Public Sector Examination.

(BHEL, NTPC, NHPC, DRDO, SAIL, HAL, BSNL, BPCL, NPCL, etc.)

HIGHWAY ENGINEERING

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CONTENT

CHAPTER 1

Introduction to Highway

DEVELOPMENT OF HIGHWAY

Roman Roads

- During this period of Roman civilization many roads were built of stone blocks of considerable thickness.

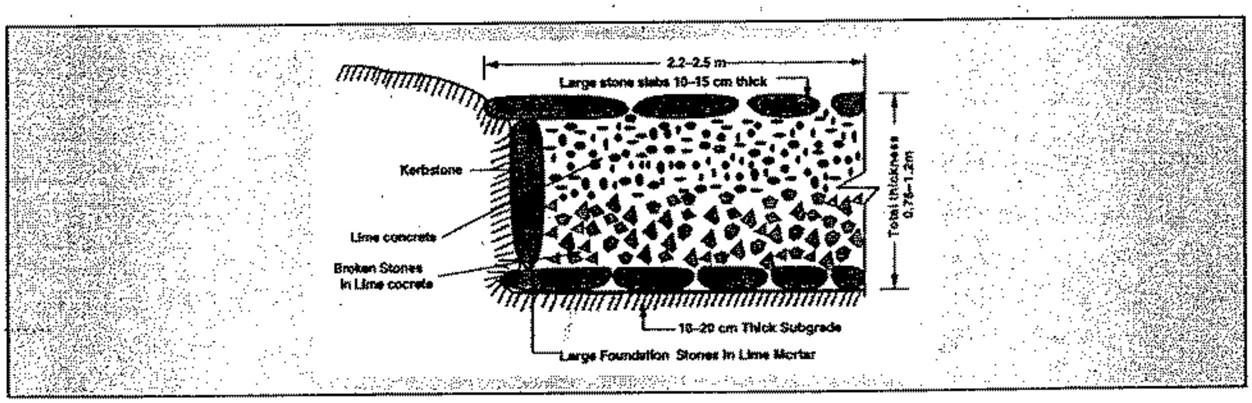
Main features of the Roman roads

(i) They were built straight regardless of gradients.

(ii) They were built after the soft soil was removed and a hard stratum was reached.

(iii) The total thickness of the construction was as high as 0.75 to 1.2 metres at some places, even though the magnitude of wheel loads of animal drawn vehicles was very low.

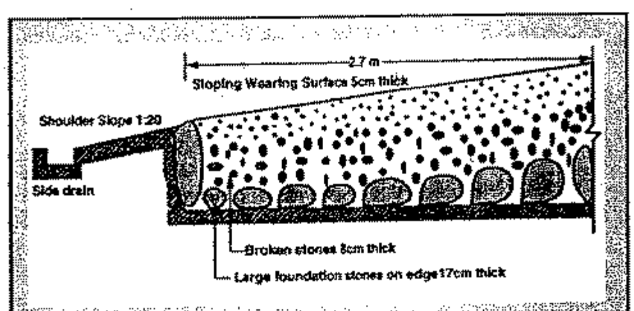
(iv) The wearing course consisted of dressed large stone-blocks set in lime mortar.



Roman roads

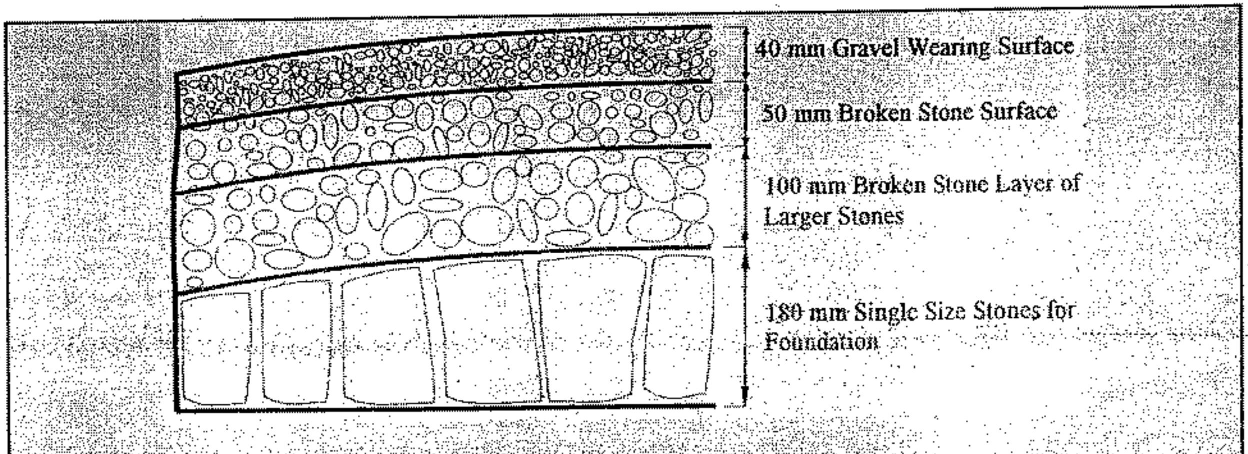
Tresaguet Construction

- Pierre Tresaguet (1716-1796) developed an improved method of construction in France by the year 1764, A.D.
- The main feature of his proposal was that the thickness of construction need to be only in the order of 30 cm.
- Due consideration was given by him to subgrade moisture condition and drainage



Telford Construction

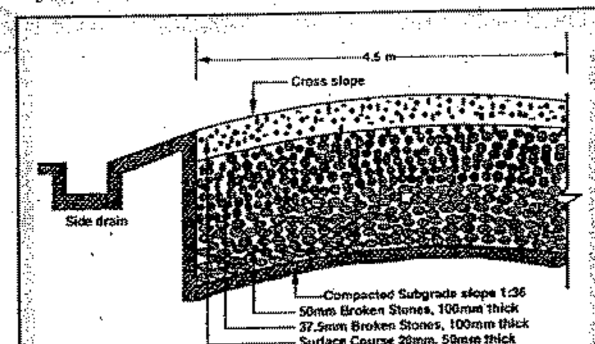
- Telford provided level subgrade of width 9 meters
- A binding layer of wearing course 4 cm thick was provided with cross slope of 1 in 45.
- Thickness of foundation stone varied from 17 cm at edges to 22 cm at the centre.



Telford's Cross-section of Road

Macadam Construction

- John Macadam (1756–1836) put forward an entirely new method of road construction as compared to all the previous methods.
- Macadam was the first person who suggested that heavy foundation stones are not at all reqd. to be placed at the bottom layer. He provided stones of size less than 5 mm to a uniform thickness of 10 cm.
- The importance to subgrade drainage and compaction was given so the subgrade was compacted and prepared with cross slope of 1 in 36.



British Roads

- The size of broken stone for the top layers was decided on the basis of stability under animal drawn vehicles.
- The pavement surface was also given the cross slope of 1 in 36.
- Total thickness was kept uniform form edge to centre to a minimum value of 25 cm.

EARLY DEVELOPMENTS IN HIGHWAY PLANNING IN INDIA

JAYAKAR COMMITTEE

- Since 1853, rail transport was mainly used for long distance transportation, and roads acted as a feeder service to the railways. After First World War, as the motor cars came on the roads, the inadequacy of the existing road network came into light.

- Hence, in 1927, the government appointed a Road Development Committee headed by Mr. M.R. Jayakar.

Recommendations of the Jayakar Committee

- Committee suggested that Central government should take the proper charge of road considering it as a matter of national interest.
- They gave more stress on long term planning programme, for a period of 20 years (hence called **twenty year plan**).
- They suggested the holding of periodic road conferences to discuss about road construction and development. This paved the way for the establishment of a semi-official technical body called **Indian Road Congress (IRC) in 1934**.
- They suggested imposition of additional taxation on motor transport which included duty on motor spirit, vehicle taxation, license fees for vehicles plying for hire. This led to the introduction of a development fund called **Central road fund (CRF) in 1929**. Road development in the country was beyond the financial capacity of local governments, and therefore, the central revenue should support it.
- They suggested a dedicated research organization to carry out research and development work. This resulted in the formation of **Central Road Research Institute (CRRI) in 1950**.

INDIAN ROADS CONGRESS

The Indian Roads Congress (IRC) was established in 1934 with the following objectives:

- To promote and encourage the science and practice of road building and maintenance.
- To provide a forum for expression of collective opinion of its members on matters affecting roads.
- To promote the use of standard specifications and practices.
- To advise regarding education, experiment and research connected with roads.
- To hold periodic meetings to discuss technical questions regarding roads.
- To suggest legislation for the development, improvement and protection of roads.
- To suggest improved methods of administration, planning, design, operation, use and maintenance of roads.
- To establish, furnish and maintain libraries and museums for furthering the science of road making.

THE NAGPUR PLAN (1943-63)

- Due to the overall economic depression in the country after First World War, the Road Development Fund was not used for development work, but spent in routine maintenance. This caused further deterioration of the roads under the impact of the heavy war time traffic.
- At this point of time, Conference of Chief Engineers of the Provinces was convened at Nagpur in 1843.
- A long-term road development plan for India was drawn up and was known as the Nagpur Plan.

Features of Nagpur Plan

- In the Nagpur plan, roads were divided into four classes:
  - (i) **National Highways**, which would traverse several provinces or states and would be of national importance for strategic, administrative and other purposes.
  - (ii) **Provincial and State Highways** which would be the other main roads of a province or state.
  - (iii) **District Roads**, which would take traffic from the main roads to the interior of each district or similar units. According to their importance, some of these are to be considered Major **District Roads** and the remaining **as Other District Roads**.
  - (iv) **Village Roads**, which would link the villages to the road system and would be designed, constructed and maintained under the authority of the provincial or state highway departments.
- National Highway would be the framework within which the road system of the country should be developed and the financial responsibility of the Centre will have an effective say in the use and control of these roads.
- National and Provincial Highways and Major District Roads would be provided with a hard durable crust.
- The committee planned to construct 2 lakh kms of road across the country within 20 years.
- They recommended the construction of star and grid pattern of roads throughout the country.
- They recommended that road length should be increased so as to give a road density of 16 kms per 100 sq.km.
- The Nagpur Plan laid down the following formulae for road length of different classes, considering the geographical, agricultural and population conditions:

(I) Length of National and Provincial Highway and Major District Roads, (in miles)

$$= \frac{A}{5} + \frac{B}{20} + N + 5T + D - R$$

where,

A = Agricultural area of province in sq. miles.

B = Non-agricultural area in sq. miles

N = Number of towns and villages having a population of 2,000-5,000

T = Number of towns and villages having a population of over 5,000,

D = An allowance for agricultural and industrial development (about 15%)

R = Railway mileage in the area under consideration.

(II) Length of other District and Village Roads, (in miles)

$$= \frac{V}{5} + \frac{Q}{2} + R + 2S + D$$

where,

Q = Number of villages with population 501-1000,

R = Number of villages with population 1,001-2,000,

S = Number of villages with population 2,001-5,000, and

D = An allowance for agricultural and industrial development during the next 20 years.

THE BOMBAY PLAN (1961-81)

- By the end of the Nagpur plan, the length of roads envisaged under it was achieved, but the road system was deficient in many respects. The changed economic, industrial and agricultural conditions in the country in that period needed a review of the Nagpur plan.
- Hence, a second long-term plan of 20-year was drafted by the Roads wing of Government of India, which is popularly known as the Bombay plan.

Features of Bombay Plan

- The total road length targeted to construct was about 10 lakhs km which will give a road density of 32 km per 100 sq. km. 40 percent of the length would be surfaced.
- The construction of 1600 km of expressways was also included in the plan.
- Funds for highway financing should come not only from direct beneficiaries (motor vehicles), but also from those on whom indirect benefits accrue. Sources which may be tapped are betterment levy, cess on land revenue, toll projects and tax on diesel oil used for motor vehicles.
- The question of vesting authority with road engineers to remove encroachments needs to be examined.
- Traffic engineering cells should be established in each State.

THE LUCKNOW PLAN (1981-2001)

Earlier two road development plans led to 2 shortcomings: (i) Ist two plans were not conceived to meet the needs of freight & passenger movement by road (ii) The plans were not part of the total transportation plan of the country.

Features of Lucknow Plan

- Roads should be classified for India as follows:
  - (a) Primary system : (i) Expressways (ii) National Highways
  - (b) Secondary system : (i) State highways (ii) Major District Roads
  - (c) Tertiary system (Rural Roads) : (i) Other District Roads (ii) Village Roads
- Road length for the year 2001 should be 27,00,000 km giving a density of 82 km/100 sq. km.
- An all-weather road should connect all villages or groups of villages with a population of 500 and above by 2001. For villages less than a population of 500, the road network shall be so planned as to result in an all-weather road being available at a distance of less than 3 km in plain areas and 5 km in hilly terrain.
- *Expressways* should be constructed on major traffic corridors to provide speedy travel.
- National Highways should form a square grid of 100 km × 100 km.
- State Highways should be extended to serve district headquarters, sub-divisional (taluka) headquarters, major industrial centres, places of commercial interest, places of tourist attraction, major agricultural market centres and ports.
- The Major District Roads should serve and connect all towns and villages with a population of 1,500 and above.
- The other District Roads should serve and connect villages with a population of 1,000-1,500.
- Energy conservation, environmental quality of roads and road safety measures were also given due importance in this plan.
- Selection of specifications should be done on the basis of (i) their amenability to stage construction

(ii) the need to adopt appropriate technology (iii) the use of local materials (iv) the use of soil-stabilization techniques (v) the use of alternate binders (vi) the use of cement concrete pavements wherever economically feasible and (vii) the need to conserve bitumen.

Following formulae give the lengths of various classes of roads as per the above guidelines:

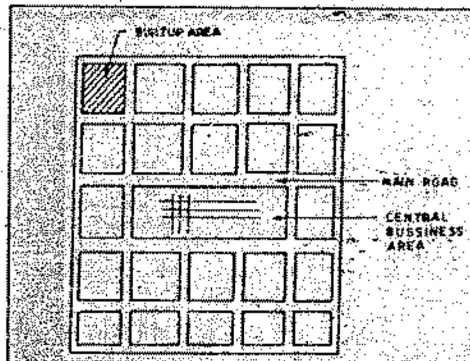
1. Length of NH (in km) $= \frac{\text{Area}}{10,000} = \frac{\text{Area(insq.km)}}{50}$
2. Length of SH (in km) $= \frac{\text{Area(in sq.km)}}{25}$;

   or Length (in km) $= 62.5 \times$ Number of towns with population above 5,000 $- \frac{\text{Area(insq.km)}}{50}$
3. Length of MDR (in km) $= \frac{\text{Area (in sq.km)}}{12.5}$

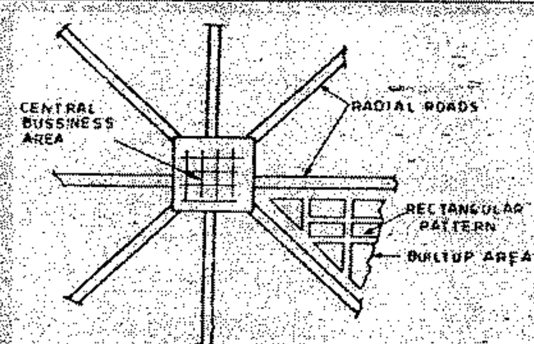
   or Length (in km) $= 90 \times$ Number of towns with population above 5,000
4. Total road length (in km) $= 4.74 \times$ Number of villages and town
5. Rural Road Length (in km) $\Rightarrow$ This can be calculated by finding the total road length and subtracting the other categories.

ROAD PATTERNS

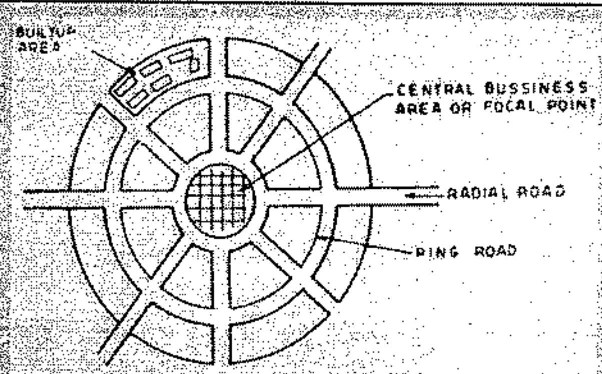
The various road patterns may be classified as follows:



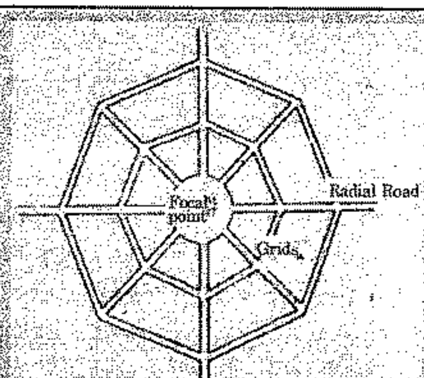
Rectangular or block pattern



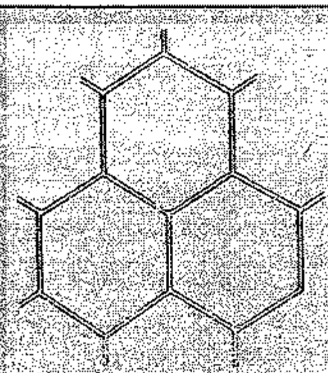
Radial or star and block pattern



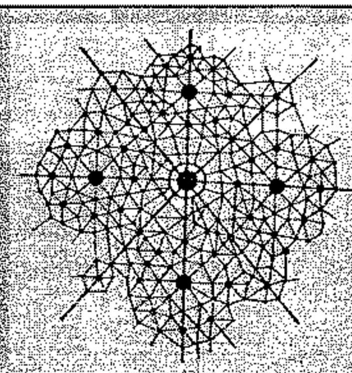
Radial or star and circular pattern



Radial or star and circular pattern



Hexagonal Patter



Minimum Travel Pattern

- The choice of the pattern very much depends on the locality, the layout of different towns, villages, industrial and production centres and on the choice of the planning engineer.
- The rectangular or the block pattern has been adopted in the city roads of Chandigarh.
- Radial and circular pattern is the road net work of Cannought Place in New Delhi.
- The Nagpur road plan formulae were prepared assuming Star and Grid pattern.

HIGHWAY ALIGNMENT AND SURVEYS

Factors Controlling Alignment

The various factors which control the highway alignment in general may be listed as :

a) Obligatory points
b) Traffic
c) Geometric design
d) Economics
e) Other considerations

a) Obligatory Points :

There are control points governing the alignment of the highways. Obligatory points through which the road alignment has to pass may cause the alignment to often deviate from the shortest or easiest path. The various examples of this category may be bridge site, intermediate town, a mountain pass or a quarry.

b) Traffic :

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

c) Geometric Design :

It may be necessary to make adjustment in the horizontal alignment of roads keeping in view the minimum radius of curve and the transition curves. Alignment should be finalised in such a way that the obstructions to visibility do not cause restrictions to the sight distance requirements.

d) Economy :

The alignment finalized based on the above factors should also be economical. The initial cost of construction can be decreased if high embankments and deep cuttings are avoided and the alignment is chosen in a manner to balance the cutting and filling.

e) Other Consideration :

Various other factors which may govern the alignment are drainage considerations, hydrological factors, political considerations and monotony.

Engineering Surveys for Highway Locations :

Before a highway alignment is finalised in highway project, the engineering survey are to be carried out. The stages of the engineering surveys are

a) Map study

b) Reconnaissance

c) Preliminary surveys

d) Final location and detailed surveys.

Map Study

By careful study of maps, it is possible to have an idea of several possible alternate routes so that further details of these may be studied later at the site. The probable alignment can be located on the map from the following details available on the map.

a) Alignment avoiding valleys, ponds or lakes.

b) When the road has to cross a new of hills, possibility of crossing through a mountain pass.

c) Approximate location of bridge site for crossing rives, avoiding bend of the river, if any.

d) When a road is to be connected between two stations, one on the top and the other on the foot of the hill, then alternate routes can be suggested keeping in view the permissible gradient.

Reconnaissance

The second stage of surveys for highway location is the reconnaissance to examine the general character of the area for deciding the most feasible routes for detailed studies. A field survey of land along the proposed alternative routs of the map in the field is studied. All relevant details not available in the map are collected and noted down. Some of the details to be collected during reconnaissance are given below :

i) Valleys, ponds, lakes, marshy land, ridge, hills permanent structures and other obstructions along the route which are not available in the map.

ii) Approximate values of gradient, length of gradients and radius of curves of alternate alignments.

iii) Number and type of cross drainage structures, maximum flood level and natural ground water level along the probable routes.

iv) Soil type along the routes.

v) Sources of construction materials.

From the details collected during the reconnaissance, the alignment proposed after study may be altered or even changed completely.

Preliminary Survey

The main objectives of the preliminary survey are :

i) To survey the various alternate alignments proposed after the reconnaissance and to collect all the necessary physical information and details of topography, drainage and soil.

ii) To compare the different proposals in view of the requirements of a good alignment.

iii) To estimate quantity of earth work and to workout the cost of alternate proposals.

iv) To finalise the best alignment from all considerations.

The procedure of preliminary survey is given in following steps :

i) Primary traverse

ii) Topographical features

iii) Levelling work.

iv) Drainage studies and Hydrological data.

v) Soil survey

vi) Material survey

vii) Traffic survey.

viii) Determination of final centre line.

Final Location and Detailed Survey

- The alignment finalized 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 finalized in the drawings is to be translated on the ground during the location survey.

Detailed Survey

- Temporary bench marks are fixed at intervals of about 250 meter and at all drainage and under pass structures. Levels along the final centre line should be taken earth work calculation and drainage details are to be worked out from the level notes. The cross section levels are taken upto the desired width, at interval of 50 to 100 meter in plain.
- The data during the detailed survey should be elaborate and complete for preparing detailed plans, design and estimates of the project.

Example 1

According to 1981 census, the area of State of Maharashtra was 308,000 sq. km. The number of towns population above 5,000 was 567. The total number of towns and villages was 35,778. Determine the length of various road categories.

Sol. Length of National Highway $= \frac{\text{Area}}{50} = \frac{308,000}{50} = 6,160$ km

Length of State Highway $= \frac{\text{Area}}{25} = \frac{308,000}{25} = 12,320$ km

or $= 62.5 \times$ No. of towns above 500 population $- \frac{\text{Area}}{50}$

$= 62.5 \times 567 - 6,160 = 35,438 - 6,160 = 29,278$ km

Length of Major District Roads Length $= \frac{\text{Area}}{12.5} = \frac{308,000}{12.5} = 24,640$ km

Length $= 90 \times$ Number of towns and villages with population above 5000

$= 90 \times 567 = 51,030$ km

Length of Total road = 4.74 × Number of towns and villages
= 4.74 × 35,778 = 1,69,587 km
Length of Rural Road = 1,69,587 − (6,160 + 12,320 + 24,640)
= 1,69,587 − 43,120 = 1,26,467 km

Note: Length of SH & MDR obtained from the Ist formulae have been used to calculate the length of rural roads. Length could be slightly different if IInd formulae for SH & MDR lengths are used.

Example 2

Determine the (1) Length of National Highways, State Highways and Major District Roads and (2) length of Other District Roads and Village Roads for a State with the following details as per the Nagpur Plan.

1. Agricultural Area in sq km = 40,000
2. Non-agricultural area in sq km = 20,000
3. Number of towns and village

Population over 5000	:	50
Population 200 – 5000	:	75
Population 100 – 2000	:	120
Population 501 – 1000	:	2000
Population less than 500	:	10000

4. Length of Railways = 1500 km

Sol. Length of NH, SH, MDR $= \frac{A}{5} + \frac{B}{20} + N + 5T + D - R$

The above formula is in FPS system. The length in km is given by suitably changing the formula as under

$$\text{Length (km)} = \frac{A \times 1.6}{(1.6 \times 1.6)5} + \frac{B \times 1.6}{(1.6 \times 1.6)20} + 1.6N + 5 \times 1.6T + D - R$$

where A and B are in sq. km.

Substituting the given values,

$$\text{Length of HN, SH and MDR} = \frac{40{,}000 \times 1.6}{1.6 \times 1.6 \times 5} + \frac{20{,}000 \times 1.6}{1.6 \times 1.6 \times 20} + 1.6 \times 75 + 5 \times 1.6 \times 50 + D - R$$
$$= 5000 + 625 + 120 + 400 + D - R$$

Assume D = 15%

$$\text{Length} = 6145 + 922 - R = (7067 - R)\text{ km} = 7067 - 1500$$
$$= 5567 \text{ km}$$

$$\text{Length of ODR and VR} = \frac{V}{5} + \frac{Q}{2} + R + 2S + D$$

This formula gives length in miles. The length in km is given by

$$= 1.6\left(\frac{V}{5} + \frac{Q}{2} + R + 2S\right) + D$$

$$= 1.6\left(\frac{1000}{5} + \frac{2000}{2} + 120 + 2 \times 75\right) + D$$

$$= 3200 + 1600 + 192 + 240 + D = 5232 + D$$

Taking D = allowance for future development as 15%

Length of ODR and VR = 5232 + 785 = 6017 km

OBJECTIVE QUESTIONS

1. **Assertion (A):** Level grades may be used in fill sections for roads in rural areas.
   Reason (R): On fill sections in rural areas, crowned pavements and sloping shoulders can take care of surface drainage.

2. The length of National Highways as per 3rd 20 year (Lucknow) road plan is given by
   (a) area of the country/75 (b) area of the country/50
   (c) area of the country/40 (d) area of the country/25

3. Aprroximate length of National highway in India is:
   (a) 1000 km (b) 5000 km
   (c) 10000 km (d) 50000 km

4. The most accessible road is
   (a) National highway (b) State highway
   (c) Major District road (d) Village road

5. In which one of the following location surveys of the road soil profile is sampling done up to a depth of 1 m to 3 m below the existing ground level?
   (a) Preliminary survey (b) Final location survey
   (c) Construction survey (d) Material location survey

6. Under the Nagpur Road Plan which of the following are NOT relevant in planning the road development programme in a backward district?
   1. Existing agriculture drainage network of drain canals.
   2. Existing number of Panchayat unions
   3. Existing of villages mud-track roads
   4. Number of villages with population of 10000 and above
   (a) 1, 2, 3 and 4 (b) 1, 2 and 3 only
   (c) 1, 2 and 4 only (d) 2, 3 and 4 only

7. The Star and Grid pattern of road network was adopted in
   (a) Nagpur Road Plan (b) Lucknow Road Plan
   (c) Bombay Road Plan (d) Delhi Road Plan

8. Three new roads P, Q and R are planned in a district. The data for these roads are given in the table below.

Road	Length (km)	Number of villages with population		
		less than 2000	2000-5000	more than 5000
P	20	8	6	1
Q	18	19	8	4
R	12	7	5	2

Based on the principle of maximum utility, the order of priority for these three roads should be

(a) P, Q, R
(b) Q, R, P
(c) R, P, Q
(d) R, Q, P

ANSWERS

1. (a)	2. (b)	3. (d)	4. (d)	5. (a)	6. (a)	7. (a)	8. (d)

CHAPTER

2

Geometric Design

INTRODUCTION

- A highway has many visible dimensions both in the horizontal plane and in the vertical plane. The art of design of the visible dimensions is known as Geometric Design.
- Proper geometric design will help in the reduction of accidents and their serverity. Therefore, the objective of geometric design is the provide optimum efficiency in traffic operation and maximum safety at reasonable cost.
- Highway Geometric Design covers elements such as design vehicle dimensions, user characteristics, terrain, highway classification, design speed, horizontal curves, vertical curves, gradient, sight distances, cross-sectional features, junctions, interchange etc.

FACTORS CONTROLLING GEOMETRIC DESIGN

Geometric design is influenced by a number of factors such as:

(a) Road user characteristics
(b) Vehicle characteristics
(c) Safety requirements
(d) Environmental considerations
(e) Economy in construction, maintenance and operation of vehicles
(f) Topography
(g) Functional classification of roads
(h) Traffic volume and composition
(i) Design speed

Safety, environmental needs and economy are built into various elements of design. The remaining factors has been discussed as given below.

Road user characteristics

- A driver takes a certain amount of time to respond to a particular traffic situation. This can be called as reaction time.
- The action of applying break on seeing a vehicle or obstruction on the road is not an instantaneous phenomenon. But it is a time-consuming phenomenon based on the psychological process involved.
- We can call these processes as perception, intellection, emotion and volition (PIEV)
- *Perception Time* is the time required for transmission of the sensations received through eyes, ears and body to the brain and the spinal chord by the nervous system. After perception *intellection* occurs, that is the formation of new thoughts and ideas. Recalling old memories of

- Linked with these two stages is emotion, based on the situation, like fear or anger. This has vital influence on the final message or decision sent by the brain to the muscle. This actual act of taking a decision to produce action is done through *volition* time.
- The total time required for PIEV, that is, from the instant the object comes in the line of sight of the driver to the instant he arrives at a decision, say, to slow down or to overtake under normal circumstance is called *reaction time.*
- This could vary from 0.5 second for simple situations to 3 to 4 seconds for complex situation. The reaction time is affected by the condition of the driver– fatigue, disease, alcohol consumption etc., his habits, skill, judgement and environmental conditions like climate, season, weather, time of day, altitude and light.
- Driver characteristics that influence safety are vision and hearing.
- Pedestrian characteristics that influence the design of pedestrian facilities are speed and space occupied. A speed of 1.2 m/sec is commonly taken for design (AASHTO).

Vehicle Characteristics

- The width of a vehicle determines the pavement lane width
- The length of a vehicle along with its wheel-base governs the turning path
- The height of a vehicle affects the vertical clearance; the weight and axle configuration of a vehicle are vital inputs for structural design of pavements
- The maximum dimension are laid down in the Motor Vehicles Act and by the India Roads Congress (IRC: 0731980 and IRC: 52-1981).

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

Since there is large variation in the dimensions of different types of vehicles that use a road, it is necessary to specify certain "Design Vehicles". A design vehicle is one whose dimensions and weight are adopted for determining the elements of a highway design.

Functional Classification of Roads

In India, non-urban roads are classified into the following functional classes:

(a) Expressways (b) National Highways (c) State Highways
(d) Major District Road (e) Other District Roads (f) Village Roads

S. No.	Type of Road	Description
(1)	Expressways	These are superior type facilities, generally with a divided carriageway, grade separations at cross-roads and fencing. They permit only fast vehicles.
(2)	National Highways (NH)	These are main highways running through the length and breadth of the country, connecting major ports, foreign highways and capitals of State/ Union territories and large industrial and tourist centres, and including roads required for strategic movements for the defence of the country.
(3)	State Highways (SH)	These are arterial roads of a state linking district headquarters and important cities within the state and connecting them with National Highways and highways of the neighbouring states.
(4)	Major District Roads (MDR)	These are important roads within a district serving areas of production and markets and connecting these with each other or with the main highways.
(5)	Other District Roads (ODR)	These are roads serving rural areas of production and providing them with outlet to market centres taluka (tehsil) headquarters block development headquarters or other main roads.
(6)	Village Roads (VR)	These are roads connecting villages or groups of villages with each other and to the nearest road of a higher category.

Urban streets are classified as follows:

(1) Express ways	These are divided arterial highways with full or partial control of access and provided generally with grade separation at intersections. These are high-speed high volume facilities.
(2) Arterial Streets	These form the principal network for through traffic in a city, generally spaced at about 1.5 km in highly developed areas and around 8 km in sparsely developed fringes. These are generally divided facilities with fully or partially controlled access.
(3) Sub-arterial Streets	These are streets of lower order of mobility than arterial streets and are spaced at 0.5 km in highly developed areas and 3-5 km in urban fringes.
(4) Collector streets	These are intended for collecting and distributing traffic to and from local streets and feeding the arterial streets.
(5) Local Streets	These are primarily intended for access to residence, business or abutting property.

Topography

The topography of the land, through which the road passes, also known as the terrain, controls the geometric design. The following terrain types are identified as controls for design in India:

Terrain Classification

S.No.	Terrain	Percentage cross-slope of country
1	Plain	0 to 10
2	Rolling	10 to 25
3	Mountainous	25 to 60
4	Steep	Greater than 60

If cross slope is large, increase in radius of curvature of road will lead to increase in construction cost. Hence, design speed is reduced so that radius of curve reduces leading in reduction in cost of construction.

Design speed

- The maximum speed at which vehicles can continuously travel safety under favourable conditions is known as design speed.
- It may also be though of as the maximum approximate speed that will be adopted by most drivers. Choice of design speed has to be made carefully, so as to match the terrain condition and also to be acceptable to most road users.
- It is the basic parameter which determines all other geometric design feature. Design speeds for various classes of roads should be as given in table

Design speeds in India for rural highways

S. No.	Road classification	Design speed, km/h							
		Plain terrain		Rolling terrain		Mountainous terrain		Steep terrain	
		Ruling design speed	Minimum design speed	Ruling design speed	Minimum design speed	Ruling design speed	Minimum design speed	Ruling	Minimum
1.	National and State Highways	100	80	80	65	50	40	40	30
2.	Major District Roads	80	65	65	50	40	30	30	20
3.	Other District Roads	65	50	50	40	30	25	25	20
4.	Village Roads	50	40	40	35	25	20	25	20

- Normally "ruling design speed" should be the guiding criterion for correlating the various geometric design features.
- "Minimum design speed" may however, be adopted in sections where site condition, including costs, do not permit a design based on the "ruling design speed"
- The design speed should preferably be uniform along a given highway. But variations in

terrain may make changes in speed unavoidable. Where this is so, it is desirable that the design speed should not be changed abruptly, but in a gradual manner by introducing successive sections of increasing/decreasing design speed so that the road user get conditioned to the change by degrees.

- For limited access facilities like expressways, a higher value is desirable. 120 km/h is commonly adopted.

For urban streets, the design speeds adopted in India (IRC) are given in Table below:

Design speeds for Urban Roads

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

Various elements of geometrical design are

(a) cross sectional element

(b) sight distance element

(c) horizontal alignment detail

(d) vertical alignment detail.

(e) intersection features.

CROSS SECTIONAL ELEMENTS

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

Pavement surface characteristics

For safe and comfortable driving, four aspects of the pavement surface are important:

(a) The friction between the wheels and the pavement surface.

(b) Smoothness of the road surface,

(c) The light reflection characteristics of the top of pavement surface and

(d) Drainage of water.

Friction

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

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

Various factor that effect friction are

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

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

Unevennes

- Unevenness affect the vehicle operating cost, speed, riding comfort, safety, fuel consumption and wear and tear of tyres.
- **Unevenness Index** is a measure of uneveness which is the cumulative measure of vertical undulation of the pavement surface recorded per unit horizontal length of the road.
- An uneveness index value less than 1500 mm/km is considered as good, a value less than 2500 mm/km is satisfactory up to speed of 100 kmph and values greater than 3200 mm/km is considered as uncomfortable even for 55 kmph.
- Pavement surface condition is measured by *Bump Indicator* in terms of uneveness index.

Light reflection

- White roads have good visibility at night, but cause glare during day time.
- Black road has no glare during day, but has poor visibility at night when the surface is wet.
- Concrete roads has better visibility and less glare

Drainage

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

CAMBER OR CROSSFALL

Camber or Cant is the cross slope provided to raise middle of the road surface in the transverse direction to drain off rain water from road surface. The objective of providing camber area:

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

Too steep slope is undesirable because

(a) It will erode the surface.

(b) Due to too steep slope, transverse tilt of vehicles causes uncomfortable side thrust and a drag on the steering 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.

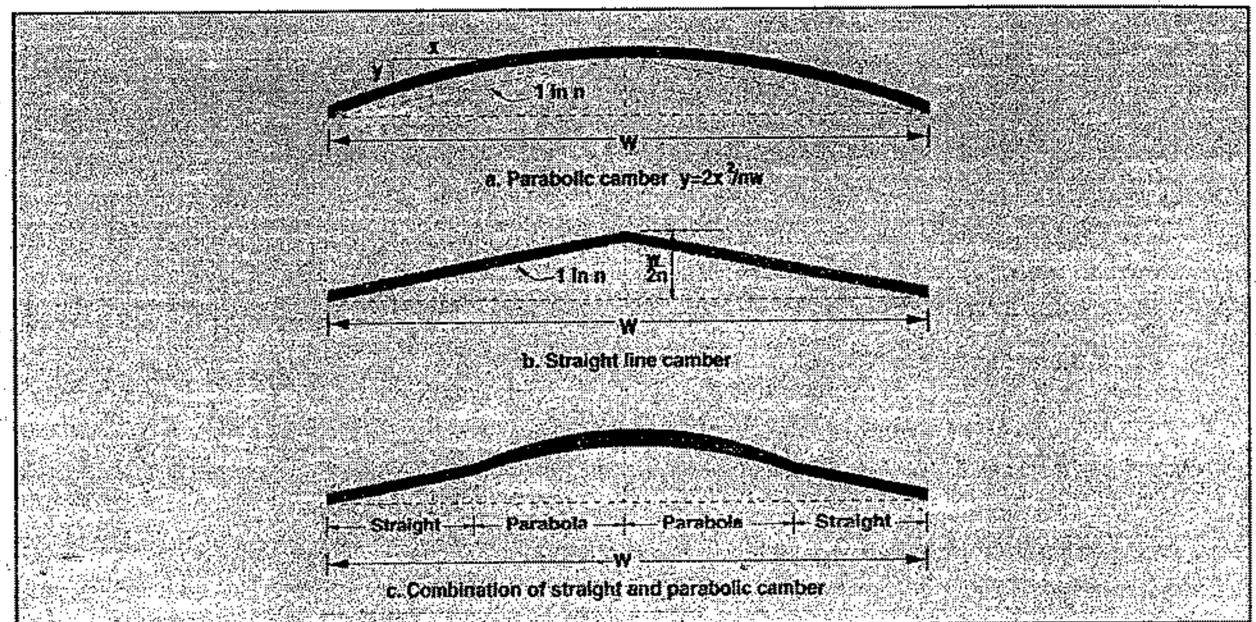
(c) Discomfort causing throw of vehicle when crossing the crown during overtaking operations.

(d) Problems of toppling over of a highly laden bullock carts and trucks.

(e) Tendency of most of the vehicles to travel along the centre line.

Shape of cross slope

The common types of camber are parabolic, straight, or combination of them



Different types of camber

- Parabolic or elliptic shape is given so that the profile is flat at the middle and steeper towards the edges, which is preferred by fast moving vehicles as they have to frequently cross the crown line during overtaking operation on a two lane highway.
- When very flat cross slope is provided as in cement concrete pavements, straight line shape of camber may be provided
- The cross slope for shoulder should be 0.5% steeper than the cross slope of adjoining pavement, subject to a minimum of 3.0%.

Providing camber in the field

For providing the desired amount and shape of camber, templates or camber boards are preferred with the specified camber.

Camber is measured in 1 *in n or n%* (Eg., 1 in 50 or 20%) and the value depends on the type of pavement surface. The values suggested by IRC for various categories of pavement is given in Table below

IRC Values for camber

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

Crossfall for Shoulders

- The crossfall for each shoulders should be at least 0.5 per steeper than the slope of the pavement subject to a minimum of 3 percent.
- On superelevated sections, shoulders should normally have the same crossfall as the pavement.

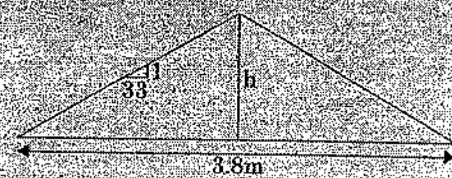
Example 1

In a district road where the rainfall is heavy major district road of WBM pavement, 3.8m wide and a state highway of bituminous concrete pavement, 7.0m wide are to be constructed. What should be the height of crown with respect to the edges in these two cases.

Sol. For WBM Road

As the rainfall is heavy, provide a camber of 1 in 33.

From Figure



$$\tan\theta = \frac{1}{33} = \frac{h}{\left(\frac{3.8}{2}\right)}$$

$\therefore$ Rise of camber with respect to Edges

$$h = \left(\frac{3.8}{2}\right)\times\frac{1}{33} = 0.058 \text{ m}$$

For Bituminous Concrete Road, provide a cross fall of 1 in 50.

Rise of crown with respect to the Edges

$$\tan\theta = \frac{h}{\left(\frac{7}{2}\right)} = \left(\frac{1}{50}\right)$$

$$h = 3.5\times\left(\frac{1}{50}\right) = 0.07\text{m}.$$

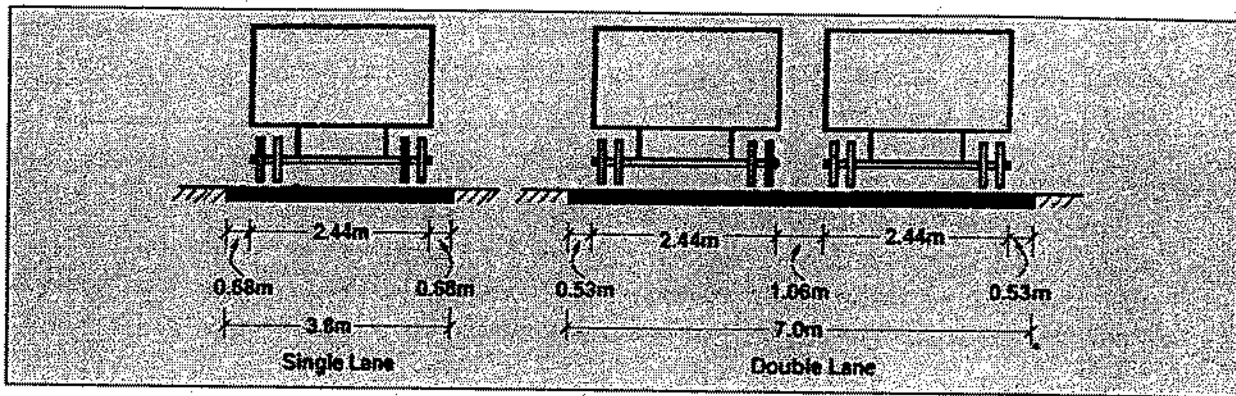
WIDTH OF CARRIAGE WAY

- Width of the carriage way or the width of the pavement depends on the width of the traffic lane and number of lanes.
- Width of a traffic-lane depends on the width of the vehicle and the clearance
- The maximum permissible width of a vehicle is **2.44** and the desirable side clearance for single lane traffic is 0.68 m. The required minimum lane width is 3.75 m for a single lane road & for double lane, however, the side clearance required is about 0.53 m, on either side on 1.06 m in the centre. Therefore a two lane road requires minimum of 3.5 m for each lane. The desirable carriage way width recommended by IRC is given in table below

IRC Specification for carriage way width

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

- Intermediate carriage way is more than one lane but less than two lane. This is provided to give manouvering.



- Where the carriageway width changes, e.g., from single lane to two lanes or two lanes to four lanes, the transition should be affected through a taper of 1 in 15 to 1 in 20.

TRAFFIC SEPARATORS OR MEDIAN

The main function of traffic separator is to prevent head-on collision between vehicles moving in opposite directions on adjacent lanes

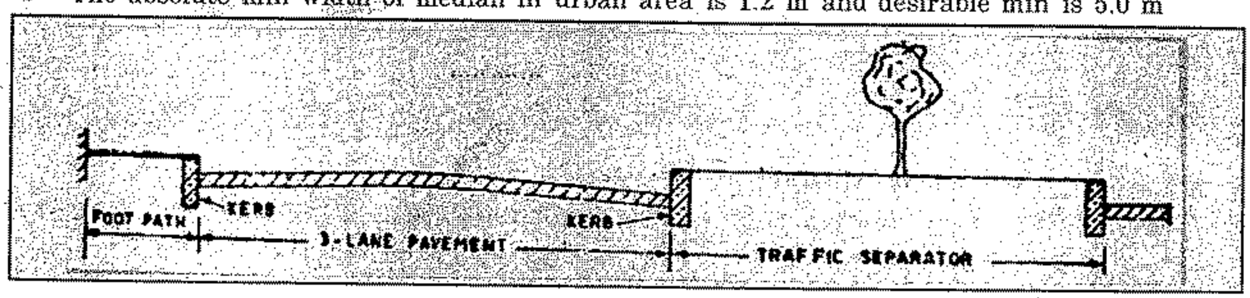
The separator may also help to

(i) Channelize traffic into streams at intersections
(ii) Shadow the crossing and turning traffic
(iii) Segregate slow traffic and to protect pedestrians.

- The traffic separators used may be in the form of pavement markings, physical dividers of area separators.
- Area separators may be medians, dividing island or parkway strips, dividing the two directions of traffic flow.

The IRC recommends a minimum desirable width of 5.0 m for median of rural highways, which may be reduced to 3.0 m where land is restricted.

- On long bridges, the width of the median may be reduced upto 1.2 to 1.5 m.
- The medians should normally be of uniform width on a particular road, but where change in width is unavoidable, a transition of 1 in 15 to 1 in 20 must be provided.
- On urban highways with six lanes or more, medians should invariably be provided
- The absolute min width of median in urban area is 1.2 m and desirable min is 5.0 m



Kerb and Traffic Separator

Shoulders

- Shoulders are provided along the road edge and is intended for accommodation of stopped vehicles, serve as an emergency land for vehicles and provides lateral support for base and surface courses.
- The shoulder should be strong enough to bear the weight of a fully loaded truck even in wet conditions.

- The shoulder width should be adequate for giving working space around a stopped vehicle. It is desirable to have a width of 4.6 m for the shoulders. A minimum width of 2.5 m is recommended for 2-lane rural highways in India.

Parking lanes

- Parking lanes are provided in urban lanes for side parking
- Parallel parking is preferred because it is safe for the vehicles moving on the road.
- The parking lane should have a minimum of 3.0 m width in the case of parallel parking.
- Parallel parking is the one in which vehicles are parked along the kerb in the longitudinal duration

Bus-bays

Bus bays are provided by recessing the kerb for bus stops. They are provided so that they do not obstruct the movement of vehicles in the carriage way. They should be atleast 75 meters away from the intersection so that the traffic near the intersections is not affected by the bus-bay.

Service Roads

- Service roads or frontage roads give access to access controlled highways like free ways and expressways
- They run parallel to the highway and will be usually isolated by a separator and access to the highway will be provided only at selected points.
- These roads are provided to avoid congestion in the expressways

Cycle track

Cycle tracks are provided in urban areas when the volume of cycle traffic is high. Minimum width of 2 m is required, which may be increased by 1 m for every additional tracks.

Footpath

- Footpaths are exclusive right of way to Pedestrians, especially in urban areas.
- They are provided for the safety of the pedestrians when both the pedestrian traffic and vehicular traffic is high.
- The footpath should be either as smooth as the pavement or more smoother than that, to induce the pedestrian to use the footpath.

Guard rails

- They are provided at the edge of the shoulder usually when the road is on an embankment
- They serve to prevent the vehicles from running off the embankment, especially when the height of the fill exceeds 3 m. Various designs of guard rails are there.
- They also give better visibility of curves at night under headlights of vehicles.

Width of formation

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

Width of formation for various class of roads

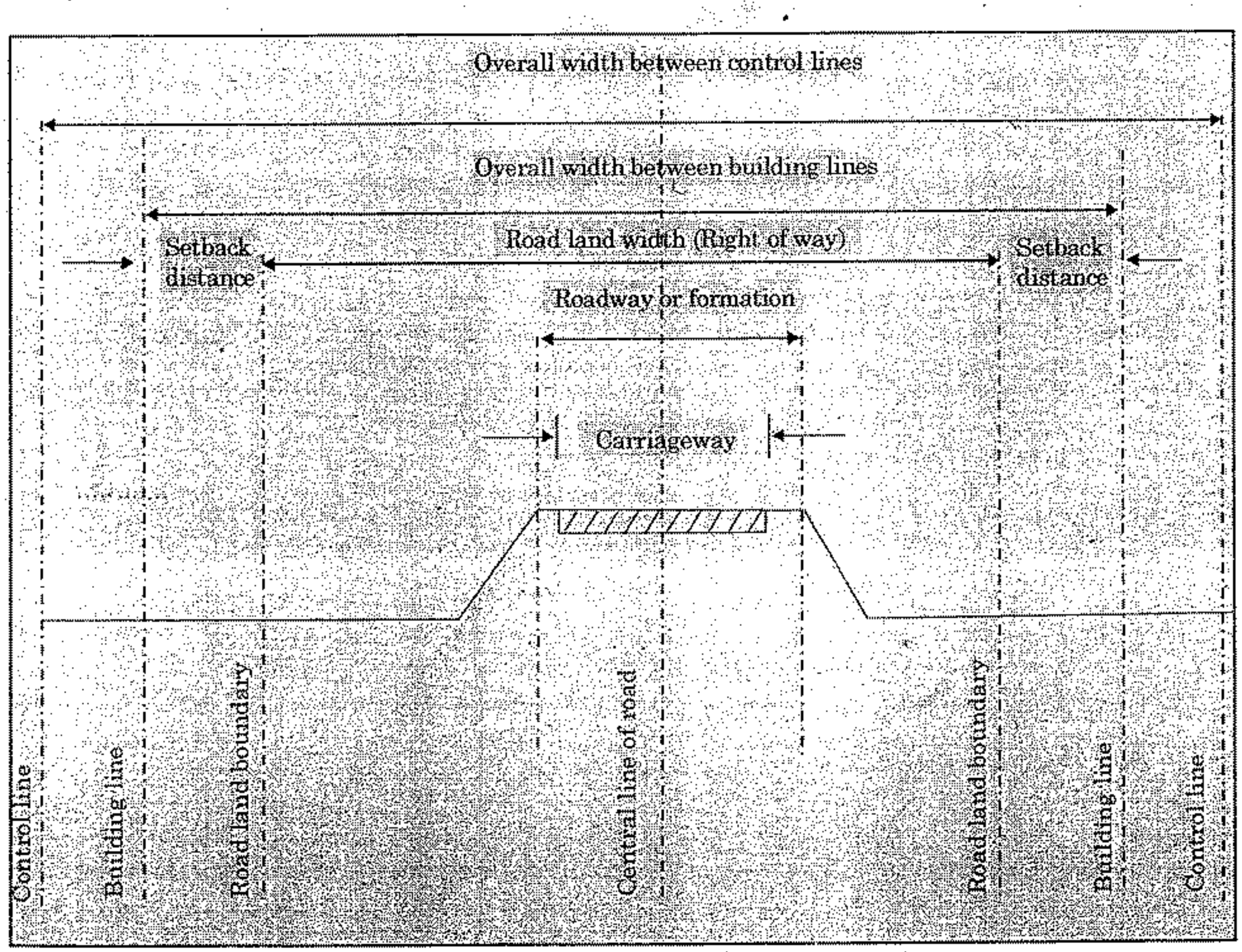
Road classification	Roadway width (in m)	
	Plain and rolling terrain	Mountainous and steep terrain
NH/SH	12	6.25-8.8 (single = 6.25, double = 8.8 m)
MDR	9	4.75 (single lane)
ODR	7.5-9.0	4.75 (single lane)
VR	7.5	4.0 (single lane)

(a) **Culverts (upto 6 m span).** In plain and rolling terrain the overall width on culverts (*measured from outside to outside of the parapet walls*) should equal the normal roadway width given in the table.

(b) **Bridges (greater than 6 m span).** At bridges, the clear width of roadway between kerb should be as under:

Single-lane bridge	...	4.25 m
Two-lane bridge	...	7.5 m
Multi-lane bridge	...	3.5 m per lane plus 0.5 m for each carriageway

(c) At causeways and submersible bridges, the minimum width of roadway (between kerbs) should be 7.5 m, unless the width is especially reduced by the competent authority.



Passing place for roads in mountains and steep terrain

Passing places or lay-byes should be provided on single lane roads in mountainous and steep terrain to cater to the following requirements:

(a) To facilitate crossing of vehicles approaching from opposite direction; and

(b) To tow aside a disabled vehicle so that it does not obstruct the traffic.

Right of way

- Right of way (ROW) or land width is the width of land acquired for the road, along its alignment.
- It should be adequate to accommodate all the cross-sectional elements of the highways and may reasonably provide for further development.
- In order to prevent overcrowding and pressure, sufficient space for future road improvement, it is advisable to lay down restrictions on building activity along the roads.
  Thus, building line represents a line on either side of the road, between which and the road no building activity is permitted at all.
- In addition, it will be desirable to exercise control on the nature of building activity for a further distance beyond the building line upto what are known as control lines.
  The right of way width is governed by:

(i) Width of formation: It depends on the category of the highway and roadway width and road margins.

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

(iii) Sight distance considerations: On curves etc. there is restriction to visibility on the inner side of the curve due to the presence of some obstruction like building etc.

(iv) Drainage system and and their size: It depends on rainfall, topography etc.

(v) Reserve land for future widening: Some land has to be acquired in advance anticipating future developments like widening of the road as land acquisition is not possible later because land may be occupied for various other purposes (buildings, business etc.). Hence, Extra width of land becomes available for the construction of roadside facilities.

Recommended land width for different classes of road

S. No.	Road classification	Plain and rolling terrain				Mountainous and steep terrain	
		Open areas		Built-up areas		Open areas	Built-up areas
		Normal	Range	Normal	Range	Normal	Normal
1.	National and State Highways	45	30-60	30	30-60	24	20
2.	Major District Roads	25	25-30	20	15-25	18	15
3.	Other District Roads	15	15-25	15	15-20	15	12
4.	Village Roads	12	12-18	10	10-15	9	9

Recommended Standards for building lines and control lines

Road classification	Plain and rolling terrain			Mountainous and steep terrain	
	Open areas		Built-up areas	Open areas	Built-up areas
	Overall width between Building Lines (metres)	Overall width between Control Lines (metres)	Distance between Building Line and road boundary (set-back) (metres)	Distance between Building Line and road boundary (set-back) (metres)	
1	2	3	4	5	6
1. National and State Highways	80	150	3-6	3-5	3-5
2. Major District Roads	50	100	3-5	3-5	3-5
3. Other District	25/30*	35	3-5	3-5	3-5
4. Village Roads	25	30	3-5	3-5	3-5

SIGHT DISTANCE

Overview

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

Types of Sight distance

- Sight distance available from a point is the actual distance along the road surface, over which a driver from a specified height above the carriage way has visibility or moving objects.
- The sight distance situations that are considered for design are :
  (i) Stopping sight distance (SSD) or the absolute minimum sight distance
  (ii) Intermediate sight distance (ISD) is defined as twice SSD
  (iii) Overtaking sight distance (OSD) for safe overtaking operation
  (iv) Head light sight distance is the distance visible to a driver during night driving under the illumination of head lights
  (v) Safe sight distance to enter into an intersection.

The computation of sight distance depends on :

(i) *Reaction time of driver*

Reaction time of a driver is the time taken from the instant the object is visible to the driver to the instant when the brakes are applied. The total reaction time may be split up into four components based on PIEV theory. IRC suggests a total reaction time of 2.5 secs. 2.5 sec. is actually the 90th percentile reaction time.

(ii) *Speed of the Vehicle*

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

(iii) *Efficiency of Brakes*

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

(iv) *Frictional Resistance between the tyre and the road*

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

(v) *Gradient of the road*

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

STOPPING SIGHT DISTANCE

- Stopping sight distance (SSD) is the minimum sight distance available on a highway at any spot having sufficient length to enable the driver to stop a vehicle travelling at design, safely without collision with any other obstruction.
- The stopping sight distance is the sum of lag distance and the braking distance.
- Lag distance is the distance the vehicle travelled during the reaction time t and is given by vt, where v is the velocity in m/sec^2.
- Braking distance is the distance travelled by the vehicle during braking operation.
- For a level road this is obtained by equating the work done in stopping the vehicle and the kinetic energy of the vehicle.
- If F is the maximum frictional force developed and the braking distance is l, then work done against friction in stopping the vehicle is $Fl = fWl$ where W is the total weight of the vehicle. The kinetic energy at the design speed is

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

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

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

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

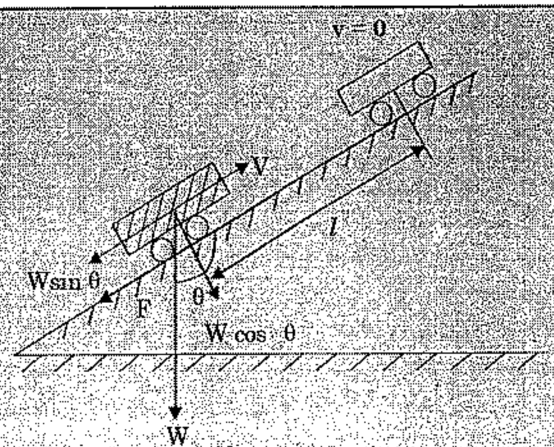
where v is the design speed in m/sec^2, t is the reaction time in sec, g is the acceleration due to gravity and f is the coefficient of friction. The coefficient of friction f is given below for variation design speed

Coefficient of longitudinal friction

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

- When there is an ascending gradient of say + n%, the component of gravity adds to braking action and hence braking distance is decreased. The component of gravity acting parallel to the surface which adds to the braking force is equal to $W\sin\theta \approx \tan\theta \approx Wn/100$.

CaseI: When vehicle is moving up the grade.

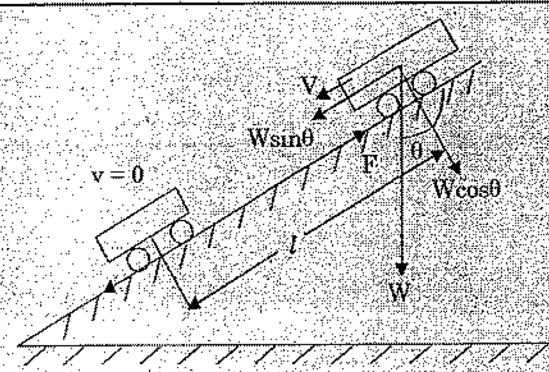


Equating kinetic energy and work done

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

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

Case II: When vehicle is moving down the grade.

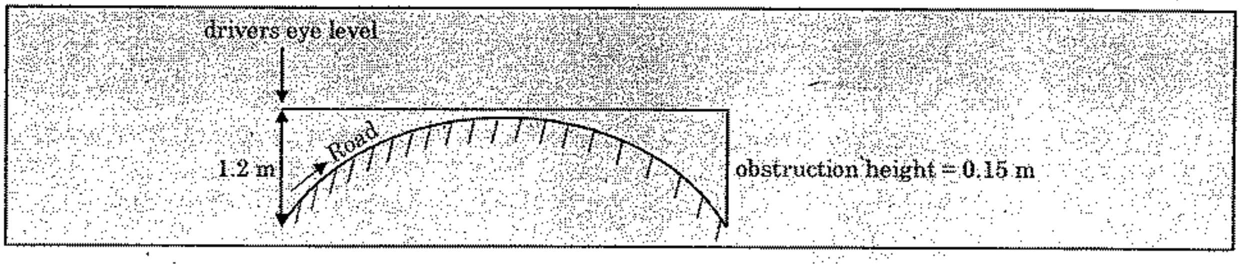


$$\boxed{SSD = vt + \frac{v^2}{2g(f - 0.01n)}}$$

Therefore the general equation is given by equation

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

- Effect for grade should not be considered for undivided highway for two way traffic but must be considered for divided highway.
- On roads with restricted width or single lane road when two-way movement of traffic is permitted, the minimum stopping sight distance should be equal to TWICE the minimum stopping distance to enable both vehicle coming from opposite directions.
- SSD on vertical curves should be the length which a driver from a height of 1.2 m have visibility of an obstruction of height 0.15 m.

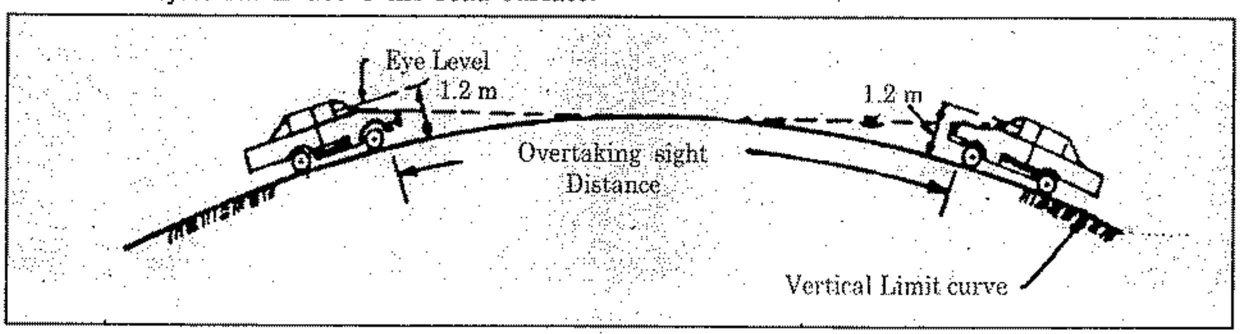


- When the stopping sight distance for the design speed is not available on any section of a road, the speed should be restricted by a warning sign and a suitable speed-limit regulation sign.

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

OVERTAKING SIGHT DISTANCE

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



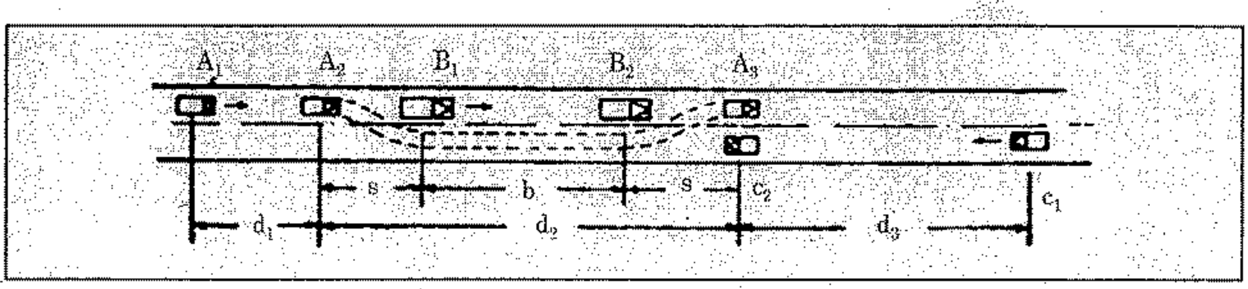
The factors that affect the OSD are:

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

Analysis of Overtaking sight distance

Figure below shows the overtaking manouevre of vehicle A travelling at design speed, and another slow vehicle B on a two-lane road with two-way traffic. Third vehicle C comes from the opposite direction

- The overtaking manoeuver may be split up into three operation, thus dividing the overtaking sight distance into three parts, d_1, d_2 and d_3.
  (i) d_1 is the distance travelled by the vehicle A during the reaction time *t* sec of the driver from position A_1 to A_2.
  (ii) d_2 is the distance travelled by the vehicle A from A_2 to A_3 during the actual overtaking operation in time T sec.
  (iii) d_3 is the distance travelled by on-coming vehicle C from C_1 to C_2 during the overtaking operation of A, *i.e*, t secs.



- Certain assumptions are made in order to calculate the values of d_1, d_2 and d_3.

In figure, A is the overtaking vehicle originally travelling at design speed v m/sec, or V kmph; B is the overtaking or slow moving vehicle move with uniform speed v_b m/sec or v_b kmph; C is a vehicle coming from opposite direction at the design speed v m/sec or V kmph. In a two-lane road the opportunity to overtake depends on the frequency of vehicles from the opposite direction and the overtaking sight distance available at any instant.

(i) It may be assumed that the vehicle A is forced to reduce its speed to the speed v_b of the slow vehicle B and moves behind it allowing a space 'S', till there is an opportunity for safe overtaking operation. The distance travelled by the vehicle A_1 during this reaction time is d_1 and is between the position A_1 and A_2. This distance will be equal to $v_b \times t$ metre where 't' is the reaction time of the driver in second.

- This reaction time 't' of the driver may be taken as two second as an average value, as the aim of the driver is only to find an opportunity to overtake. Thus,

$$\boxed{d_1 = v_b \times t = 2v_b}$$

(ii) From position A_2, 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 A_3 in time T sec. The straight distance between position A_3 in time T sec. The straight distance between position A_2 and A_3 is taken as d_2. The minimum distance between position A_2 and B_1 may be taken as the minimum spacing 's' of 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:

$$\boxed{s = (0.7v_b + 6), m}$$

- The minimum distance between B_2 and A_3 may also be assumed equal to s as mentioned above. If the time taken by vehicle A for the overtaking operation from position A_2 to A_3 is T second, the distance covered by the slow vehicle B travelling at a speed of v_b m/sec. = b = v_bT, m

Thus the distance d_2 = (b + 2s), m

- Now the time T depends on speed of overtaken vehicle B and the acceleration of overtaking vehicle A. This time T may be calculated by equating the distance d_2 to $\left(v_b T + \frac{1}{2} a\, T^2\right)$, using the general formula for the distance travelled by an uniformly accelerating body with initial speed v_b m/sec and 'a' is the acceleration in m/sec^2

$$d_2 = (b + 2s) = \left(v_b T + \frac{aT^2}{2}\right)$$

$$b = v_b \cdot T, \text{ and therefore } 2s = \frac{aT^2}{2}$$

Therefore, $T = \sqrt{\frac{4s}{a}}$ sec, where $s = (0.7\, v_b + 6)$

Hence, $d_2 = (v_b\, T + 2s)$, m

(iii) 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 is the distance d_3 between position C_1 to C_2.

Hence, $d_3 = v \times T$

Thus the overtaking sight distance,

$$OSD = (d_1 + d_2 + d_3) = (v_b t + v_b T + 2s + vT) \quad \text{... (i)}$$

In kmph units, equation (i) worksouth as:

$$OSD = 0.28\, V_b t + 0.28\, V_b\, T + 2s + 0.28\, V.T \quad \text{... (ii)}$$

Here,

v_b = speed of overtaken vehicle, kmph

t = reaction time of driver = 2secs.

V = speed of overtaking vehicle of design speed, kmph

$$T = \sqrt{\frac{4 \times 3.6s}{A}} = \sqrt{\frac{014.4s}{A}}$$

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

A = acceleration, kmph/sec.

- In case the speed of overtaken vehicle V_b is not given, the same may be assumed as (V – 16) kmph where V is the design speed in kmph or $v_b = (v - 4.5)$ m/sec and v is the design speed in m/sec.
- The acceleration of overtaking vehicle is to be specified. Usually this depends on the make of the vehicle, its condition, load and the speed.
- The average rate of acceleration during overtaking maneuver may be taken corresponding to the design speed.
- At overtaking sections, the minimum overtaking distance should be $(d_1 + d_2 + d_3)$ when two -way traffic exists.
- On divided highways and on roads with one way traffic regulation, the overtaking distance need to be only $(d_1 + d_2)$ as no vehicle is expected from the opposite direction.
- On divided highways with four or more lane, IRC suggests that it is not necessary 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).

Maximum overtaking acceleration at different speeds

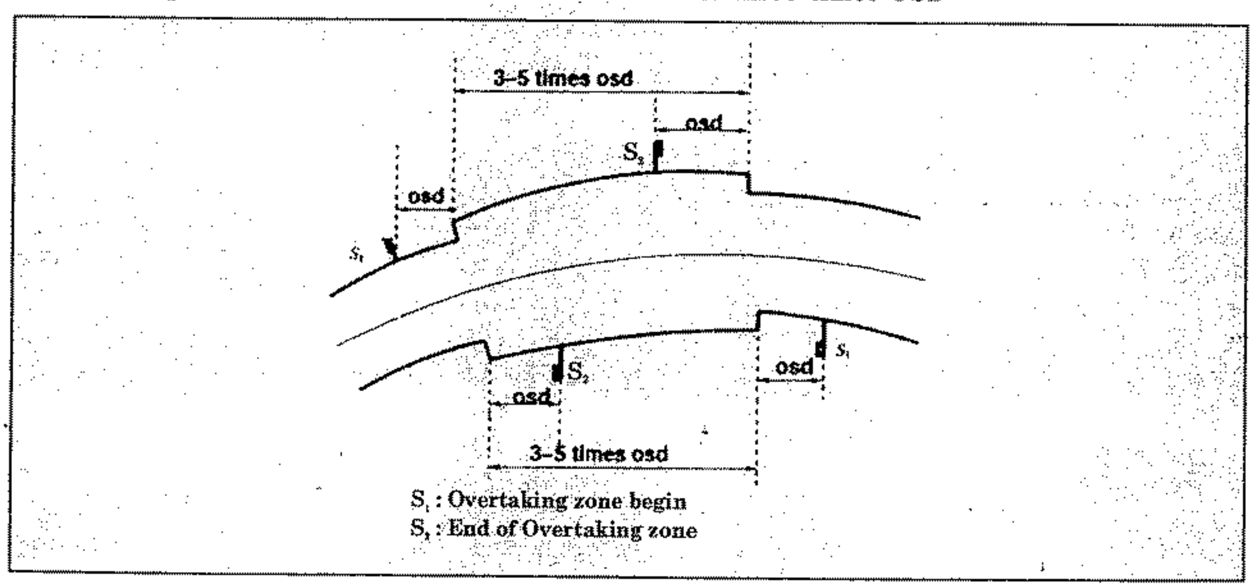
Speed		Maximum overtaking acceleration	
V (kmph)	v (m/sec.)	A (kmph/sec)	a (m/sec^2)
25	6.93	5.00	1.41
30	8.34	4.80	1.30
40	11.10	4.45	1.24
50	13.86	4.00	1.11
65	18.00	3.28	0.92
80	22.20	2.56	072
100	27.80	1.92	0.53

Effect of grade in overtaking sight distance

On upgrades, the sight distance required would be more due to reduced acceleration of the overtaking vehicle and the likely speeding up of the vehicle from opposing direction. These factors are somewhat compensated by the loss in speed of the overtaken vehicle which may frequently be a heavy truck. No separate design are, therefore, recommended for application on grades.

Overtaking zones

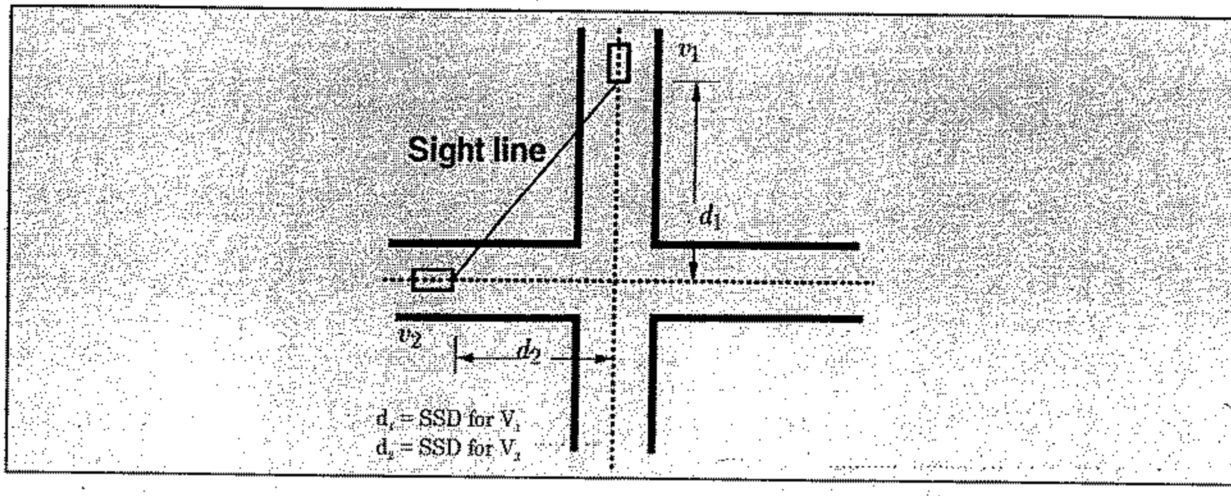
Overtaking zones are provided when OSD cannot be provided throughout the length of the highway. These are zones dedicated for overtaking operation, marked with wide roads. The desirable length of overtaking zones is 5 time OSD and the minimum is three times OSD



Overtaking zones

Sight distance at intersections

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



Sight distance at intersections

Example 2

Calculate the safe stopping sight distance for design speed of 50 km/hr for

(a) Two way traffic on a two lane road

(b) Two way traffic on a single lane road

Assume coefficient of friction = 0.37, reaction time for driver = 2.5 sec.

Sol.

$$\text{S.S.D} = \left(0.278vt_R + \frac{(0.278v)^2}{2g(f \pm n\%)}\right)$$

$$= (\text{Lag distance} + \text{Braking distance})$$

Given Data,

Design speed v = 50 km/hr

Reaction time t_R = 2.5 sec.

Coefficient of friction = f = 0.37

(a) For two way traffic with two lane

$$\text{S.S.D} = (0.278 \times 50 \times 2.5) + \frac{(0.278 \times 50)^2}{2 \times 9.81 \times (037 \pm 0)} = 61.4 \text{ m.}$$

(b) For two way traffic with single lane

$$\text{S.S.D} = 2 \times 61.4 = 122.8\text{m.}$$

Example 3

Calculate the minimum sight distance required to avoid a head on collision of two cars approaching from opposite directions at 90 and 60 km/hr. Assume a reaction time of 2.5 seconds. Coefficient of friction of 0.7 and a brake efficiency of 50% in either case.

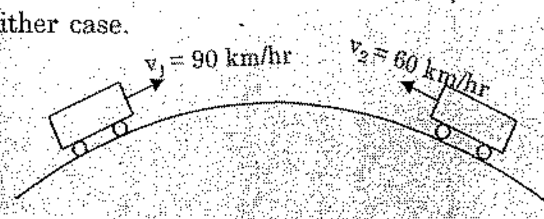
Sol. Given Data,

v_1 = 90 km/hr

v_2 = 60 km/hr

Reaction time t_R = 2.5 sec.

Brake Efficiency = 50%



Note : As the Braking Efficiency is 50% the wheel will skid through 50% of the braking distance and rotate through the remaining distance therefore the value of coefficient of friction

$$f = (0.50 \times 0.70) = 0.35$$

Therefore the stopping distance for the first case :

$$SD_1 = \left(0.278 v_1 t_R + \frac{(0.278 v_1)^2}{2g \times (f \pm n\%)}\right)$$

$$= \left(0.278 \times 90 \times 2.5 + \frac{(0.278 \times 90)^2}{2 \times 9.81 \times 0.35 \pm 0}\right)$$

$$= 153.6 \text{ m} \qquad \text{...(i)}$$

The stopping sight distance for the second case

$$SD_2 = \left(0.278 \times v_2 \times t_R) + \frac{(0.278 \times v_2)^2}{2 \times 9.81 \times (f \pm n\%}\right)$$

$$= (0.2780 + 60 \times 2.5) + \frac{(0.278 \times 60)^2}{(2 \times 9.81 \times 0.35)} = 82.2 \text{ m}$$

$\therefore$ Sight distance to avoid head on collision of the two approaching car.

$$= (SD_1 + SD_2) = (153.6 + 82.2) = 235.8 \text{ m}$$

Example 4

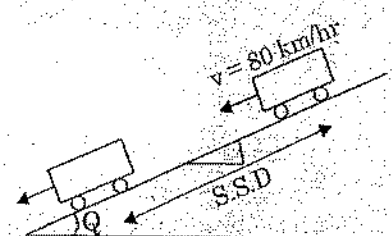
Calculate the stopping sight distance on a highway at a descending gradient of 2% for a design speed of 80 km/hr. Assume other data as per IRC recommendations.

Sol. As per IRC

Total reaction time may be taken as 2.5 sec. and design coefficient of friction as $f = 0.35$.

Given data, $n = -2\%$

Design speed speed, $v = 80$ km/hr



$$\therefore \quad S.S.D = (0.278 v t_R) + \frac{(0.278 \times v)^2}{2g \times (f \pm n\%)} \cdots$$

$$= (0.278 \times 80 \times 2.5) + \frac{(0.278 \times 80)^2}{\left(2 \times 9.81 \times \left(0.35 - \frac{2}{100}\right)\right)}$$

$$= (55.5 + 76.2) = 131.7 \text{ m.} \approx 132 \text{ m.}$$

Example 5

Calculate the values of

(i) Head light sight distance and

(ii) Intermediate sight distance for a highway with a design speed of 65 km/hr. Assume suitably all the data required.

Sol. Data given,

Design speed, v = 65 km/hr; Assume f = 0.36; t_R = 2.5 sec.

(i) Head light sight distance $= \text{S.S.D} = (0.278\text{v} \times t_R) + \dfrac{(0.278 \times \text{v})^2}{2\text{g} \times (\text{f} \pm \text{n}\%)}$

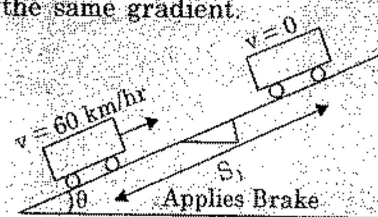
$$= (0.278 \times 65 \times 2.5) + \frac{(0.278 \times 65)^2}{(2 \times 9.81 \times 0.36)} = 91.4 \text{ m.}$$

(ii) Intermediate sight distance (I.S.D) = 2 × (S.S.D) = 2 × 91.4 = 182.8 m

Example 6

The driver of a vehicle travelling 60 km/hr up a gradient requires 9m less to stop after he applies brakes, as compared to a driver travelling at same speed, down the same gradient.

Given, f = 0.40. What is the present gradient.



Sol. Data given,

Speed, v = 60 km/hr; f = 0.40; $S_1 = (S_2 - 9)$

$\therefore$ Braking dist. up the gradient, $S_1 = \dfrac{(0.278 \times \text{v})^2}{2\text{g}(f + \text{n}\%)}$

$$= \frac{(0.278 \times 60)^2}{2 \times 9.81 \times (0.40 + \text{n}\%)} \quad \text{...(i)}$$

Braking dist. down the gradient, $S_2 = \dfrac{(0.278 \times \text{v})^2}{2\text{g}(f - \text{n}\%)} = \dfrac{(0.278 \times 60)^2}{2\text{g} \times (0.40 - \text{n})}$ ...(ii)

Therefore,

$$S_1 = S_2 - 9$$

$$\Rightarrow \quad S_1 - S_2 = -9$$

$$\therefore \quad \frac{(0.278 \times 60)^2}{2 \times \text{g}} \left[\frac{1}{(0.40 + \text{n})} - \frac{1}{(0.40 - \text{n})} \right] = -9$$

$$\frac{(0.40 - \text{n} - 0.4 - \text{n})}{(0.40)^2 - \text{n}^2} = \frac{-9 \times 2 \times 9.81}{(0.248 \times 60)^2}$$

Solving for n

$$\Rightarrow \quad \text{n} = 0.05$$

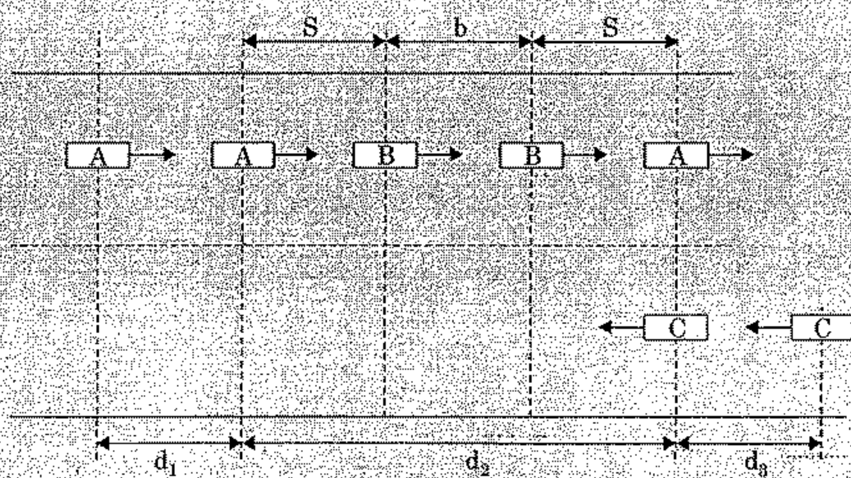
$$\text{Gradient, n} = \left(\frac{1}{20}\right)$$

Example 7

On a two-way traffic road, the speed of overtaking vehicles are 65 and 40 km/hr. If the avg. acceleration is 0.92 m/sec². Determine the overtaking sight distance indicating the details of overtaking operations.

Show the min length of overtaking zone and details of overtaking zone by a neat sketch.

Sol.



Overtaking Sight Distance for a two-way traffic = $(d_1 + d_2 + d_3)$

Given Data :

v_B = the speed of the overtaken vehicle or slow moving vehicle = 40 km/hr

v_C = Speed of the vehicle coming from opposite side = 65 km/hr.

a = Avg. Acceleration = 0.92 m/sec^2

t_R = 2.0 sec (Assume)

(i) Distance d_1 :

$\therefore$ $d_1 = (0.278 \times v_b \times t_R) = (0.278 \times 40 \times 2.0)$

$d_1 = 22.24$ m. ... (i)

(ii) Distance d_2 :

$$d_2 = 0.278\ v_b \times T + \frac{1}{2}aT^2 = (0.278\ v_b \times T + 2S)$$

where, $S = (0.2\ v_b + 6) = (0.2 \times 40 + 6) = 14$ m.

$$T = \sqrt{\left(\frac{4S}{a}\right)} = \sqrt{\frac{4 \times 14}{0.92}} = 7.8 \text{ sec.}$$

$\therefore$
$$d_2 = \left(0.278 v_b \times T + \frac{1}{2}aT^2\right)$$

$$= (0.278 \times 40 \times 7.8) + \frac{1}{2} \times 0.92 \times (7.8)^2 = 114.73\text{m.}$$

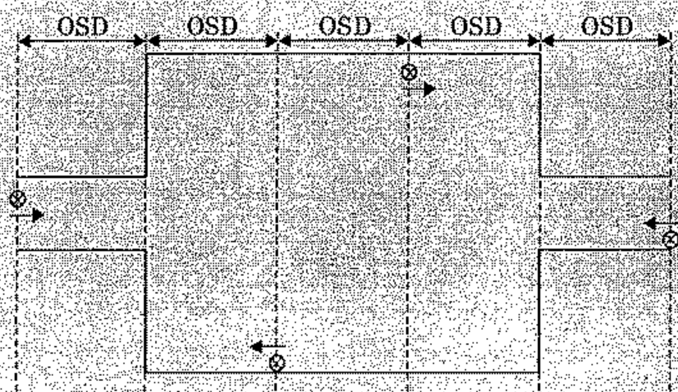
(iii) Distance d_3 :

$d_3 = (0.278 \times v_c \times T) = (0.278 \times 65 \times 7.8) = 140.95$m.

$\therefore$ Total OSD $= (d_1 + d_2 + d_3) = (22.24 + 114.73 + 140.95)$

$= 277.92$ m.

Overtaking zone



Min length of overtaking zone $= 3(\text{OSD}) = 3 \times 277.92 = 833.74$ m.

Desirable length of overtaking zone $= 5\ (\text{OSD}) = 5 \times 277.92 = 1389.6$ m

Example 8

Calculate the safe overtaking sight distance for a design speed of 96 km/hr. Assume all other data suitably.

Sol. Assume, $v_b = (v_c - 16) = 96 - 16 = 80$ km/hr

$$a = 2.5 \text{ kmph/sec} = \left(\frac{2.5 \times 5}{18}\right) \text{ m/sec} = 0.694 \text{ m/sec}^2$$

$$t_R = 2 \text{ sec.}$$

$\therefore$ O.S.D. for one-way traffic $= (d_1 + d_2)$

For two-way traffic $= (d_1 + d_2 + d_3)$

(i) Distance d_1:

$$d_1 = 0.278\ v_b t_R = 0.278 \times 80 \times 2 = 44.8 \text{ m}$$

(ii) Distance d_2:

$$d_2 = 0.278 v_b \times T + \frac{1}{2} a T^2 = (0.278\ v_b T + 2S)$$

where, $S = (0.2 v_b + 6) = (0.2 \times 80 + 6) = 22$m.

$$T = \sqrt{\frac{4S}{a}} = \sqrt{\frac{4 \times 22}{0.694}} = 11.3 \text{ sec.}$$

$$d_2 = (0.278 \times 80 \times 11.3 + \frac{1}{2} \times 0.694 \times (11.3)^2 = 297 \text{ m.}$$

(iii) Distance d_3:

$$d_3 = (v_c \times T) \times 0.278 = 0.278 \times 96 \times 11.3 = 303.7 \text{ m.}$$

$\therefore$ OSD on one-way traffic $= (d_1 + d_2) = 341.8$ m.

$\therefore$ OSD on two-way traffic Road $= (d_1 + d_2 + d_3) = 645.5$ m.

HORIZONTAL ALIGNMENT

Overview

Horizontal alignment is one of the most important features influencing the efficiency and safety of a highway. A poor design will result in lower speeds and resultant reduction in highway performance in terms of safety and comfort. In addition, it may increase the cost of vehicle operations and lower the highway capacity. The horizontal alignment design elements include radius of circular curve, design of superelevation, extra widening at horizontal curves, design of transition curve, and set back distance.

Design speed

The design speed, is the single most important factor in the design of horizontal alignment. The design speed depends on the type of the road, type of terrain.

Indian Road congress (IRC) has classified the terrains into four categories, namely plain, rolling mountainous, and steep based on the cross slope. The recommended design speed for various terrains and type of roads are an given in table below.

Terrain classification

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

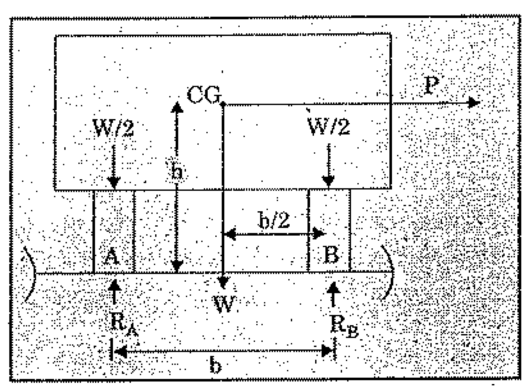
Design speed in km/hr as per IRC (ruling and minimum)

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

Horizontal curve

The presence of horizontal curve imparts centrifugal force which is a reactive force acting outward on a vehicle negotiating it. Centrifugal force depends on speed and radius of the horizontal curve and is counteracted to a certain extent by transverse friction between the tyre and pavement surface.

- On a curved road,, this force tends to cause the vehicle to overrun or to side outward from the centre of road curvature. For proper design of the curve, and understanding of the forces acting on a vehicle taking a horizontal curve is necessary. Various forces acting on the vehicle are illustrated in the figure below



These are the centrifugal force (P) acting outward, weight of the vehicle (W) acting downward, and the reaction of the ground on the wheels (R_A and R_B). The centrifugal force and the weight is assumed to be from the centre of gravity which is at h units above the ground. Let the wheel base be assumed to be b units. The centrifugal force P is given by

$$\boxed{P = \frac{Wv^2}{gR}} \quad \text{... (i)}$$

where W is the weight of the vehicle, v is the speed of the vehicle, g is the acceleration due to gravity and R is the radius of the curve

- The centrifugal ratio or the impact factor $\frac{P}{W}$ is given by:

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

- The centrifugal force has two effects: A tendency to overturn the vehicle about the outer wheels and a tendency for transverse skidding. Taking moments of the force with respect to the outer wheel when the vehicle is just about to overturn (under this condition, reactor at inner wheel will be zero).

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

At the equilibrium, over turning is possible when

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

and for safety the following condition must satisfy:

$$Ph \le \frac{Wb}{2}$$

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

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

$$\text{or} \quad \boxed{\frac{b}{2h} \ge \frac{v^2}{gR} \text{ (for no overturning)}}$$

- The second tendency of the vehicle is for transverse skidding. i.e., when the centrifugal force P is greater than the maximum possible transverse skid resistance due to friction between the pavement surface and tyre.

The maximum skid resistance (F) is given by:

$$F = F_A + F_B = f(R_A + R_B) = fW$$

where F_A and F_B is the frictional force at tyre A and B, R_A and R_B is the reaction at tyre A and B, f is the lateral coefficient of friction and W is the weight of the vehicle. This is counteracted by the centrifugal force (P), and equating:

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

At equilibrium, when skidding takes place [from equation (ii)]

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

and for safety the following condition must satisfy:

$$P \leq fW$$

$$\frac{P}{W} \leq f$$

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

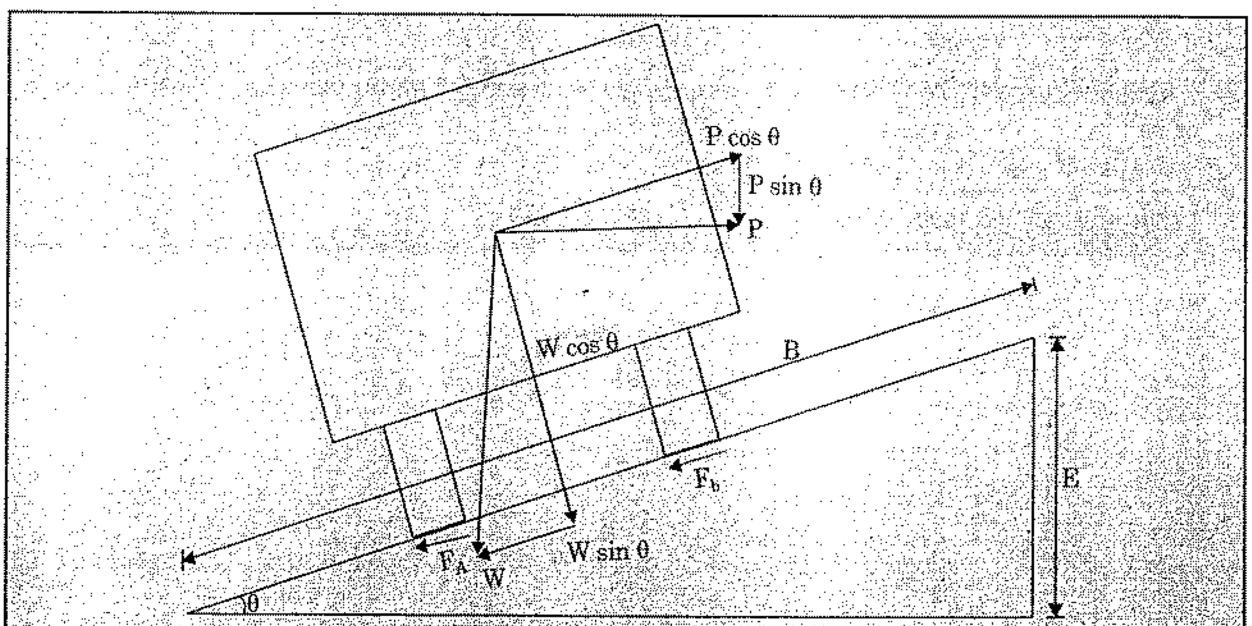
$$\boxed{f > \frac{v^2}{gR}\,(\text{for no skidding})} \quad \text{... (iv)}$$

Equation (iii) and (iv) gives the stable condition for design. If equation (iii) is violated, the vehicle will overturn at the horizontal curve and if equation (iv) is violated, the vehicle will skid at the horizontal curve. $\boxed{\text{For no sliding \& no overtuning } \frac{P}{W} \leq \frac{b}{2h} \leq f.}$

ANALYSIS OF SUPERELEVATION

- Super-elevation or cant or banking is the transverse slope provide at horizontal curve to counteract the centrifugal force, by raising the outer edge of the pavement with respect to the inner edge, throughout the length of the horizontal curve.
- When the outer edge is raised, a component of the curve weight will be complemented in counteracting the effect of centrifugal force.
- In order to find out how much this raising should be, the following analysis may be done.

  Force acting on a vehicle while taking a horizontal curve of radius R m at a speed v m/sec are:



- P is the centrifugal force acting horizontally out-wards through the center of gravity

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

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

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

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

$$\frac{P\cos\theta}{W\cos\theta} = \frac{W\sin\theta}{W\cos\theta}+\frac{fW\cos\theta}{W\cos\theta}+\frac{fP\sin\theta}{W\cos\theta}$$

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

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

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

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

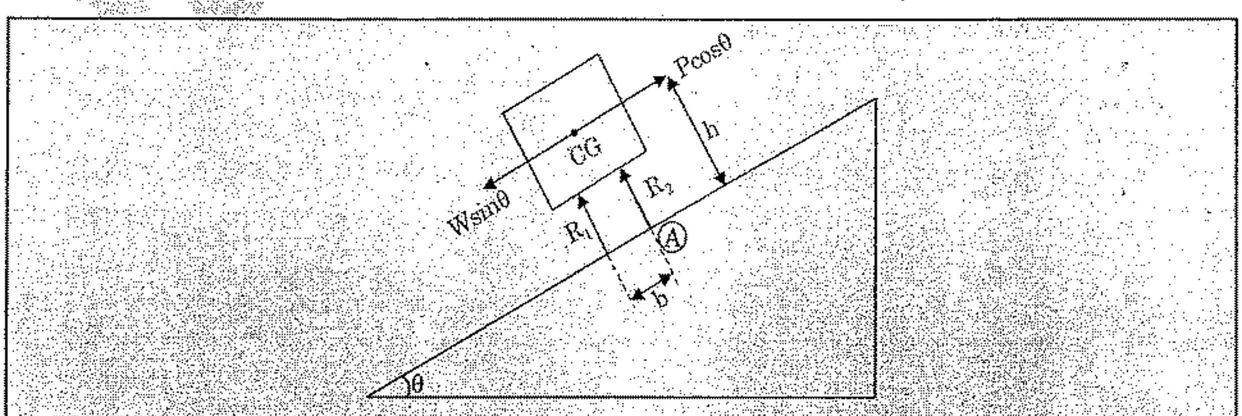
This is an exact expression for superelevation. But normally, for $f = 0.15$ and $\theta < 4°$, $1 - f\tan\theta \approx 1$ and for small , $\tan\theta \approx \sin\theta = E/B = e$, thus

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

where, e is the rate of super elevation, f the coefficient of lateral friction 0.15, v the speed of the vehicle in m/sec, R the radius of the curve in m and $g = 9.8 \text{m/sec}^2$.

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

1. If there is no friction due to some practical reasons, then $f = 0$ and equation becomes $e = \frac{v^2}{gR}$. This results in the situation where the pressure on the outer and inner wheels are same [from moment about CG = 0, we have $R_1 = R_2$] requiring very high super-elevation e $\boxed{e=\frac{v^2}{gR}}$ is called equilibrium superelevation.



$$W \sin \theta = P \cos\theta \qquad \text{[if, } f = 0\text{]}$$

$$W \cos \theta + P \sin \theta = R_1 + R_2$$

Taking moment about A

$$R_1 b - (W \cos \theta + P \sin \theta) \frac{b}{2} + P \cos\theta \times h - W \sin h = 0$$

$$\Rightarrow \qquad R_1 b - (R_1 + R_2)\frac{b}{2} = 0$$

$$R_1 = R_2$$

2. If there is no super-elevation provided due to some practical reasons, then $e = 0$ and equation becomes $f = \frac{v^2}{gR}$. This $\frac{v^2}{gR}$ results in a very high coefficient of friction.
3. If $e = 0$ and $f = 0.15$ then for safe traveling speed $v_b = \sqrt{fgR}$ where v_b is the restricted speed.

GUIDELINES ON SUPERELEVATION

- While designing the various elements of the road like superelevation, we design it for a particular vehicle called design vehicle which has some standard weight and dimensions.
- But in the actual case, the road has to cater for mixed traffic. Superelevation designed for a particular vehicle travelling at a particular speed may not be suitable for other vehicle and may even cause toppling. Taking into account these consideration, IRC has given some guidelines about the maximum and minimum superelevation etc.

Design of super-elevation

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

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

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

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

Step 4 : Find the allowable speed v_a for the maximum $e = 0.07$ and $f = 0.15$, $v_a' = \sqrt{0.22gR}$ if $v_a \geq v$ then the design is adequate and provide $e = 0.07$ otherwise speed is limited to allowable speed v_a.

- Raddi beyond which superelevation is required.

$$e_1 = \frac{(0.75v)^2}{gR},$$

if e_1 is taken as camber then $R = \frac{(0.75v)^2}{ge_1}$ is the radius beyond which no superelevation is required.

Maximum and minimum super-elevation

IRC specifies a maximum super-elevation of 7 percent for plain and rolling terrain, while that of hilly terrain (not bound by snow) is 10 percent and urban road is 4 percent. The minimum super elevation is 2-4 percent for drainage purpose, especially for large radius of the horizontal curve. For hilly terrain bound by snow we should take max super elevation as 7%.

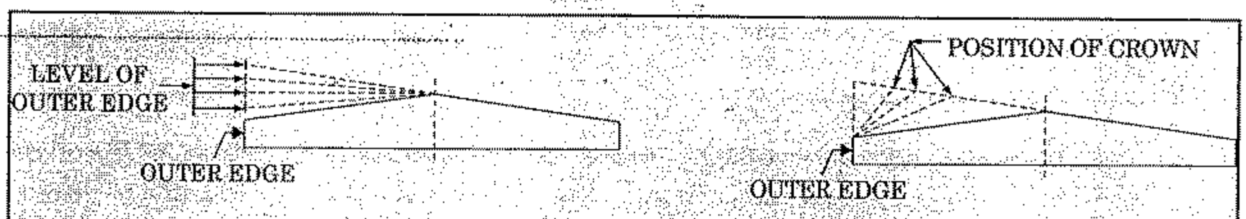
Attainment of super-elevation

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

(a) Elimination of crown of the cambered section

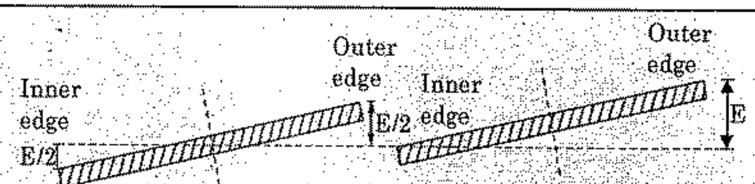
(b) Rotation of pavement to attain full superelevation

1. Elimination of the crown of the cambered section by:
   (a) rotating the outer edge about the crown. The outer half of the cross slope is rotated about the crown at a desired rate such that this surface falls on the same plane as the inner half.
   (b) shifting the position of the crown: This method is also known as diagonal crown method. Here the position of the crown is progressively shifted outwards, thus increasing the width of the inner half of cross section progressively.



(a) Outer edge rotated about the crown (b) Crown shifted outwards (diagonal crown method)

2. Rotation of the pavement cross section to attain full super elevation by: There are two methods of attaining superelevation by rotating the pavement.
   (a) rotation about the center line: The pavement is rotated such that the inner edge is depressed and the outer edge is raised both by half the total amount of superelevation, i.e., by E/2 with respect to the centre.



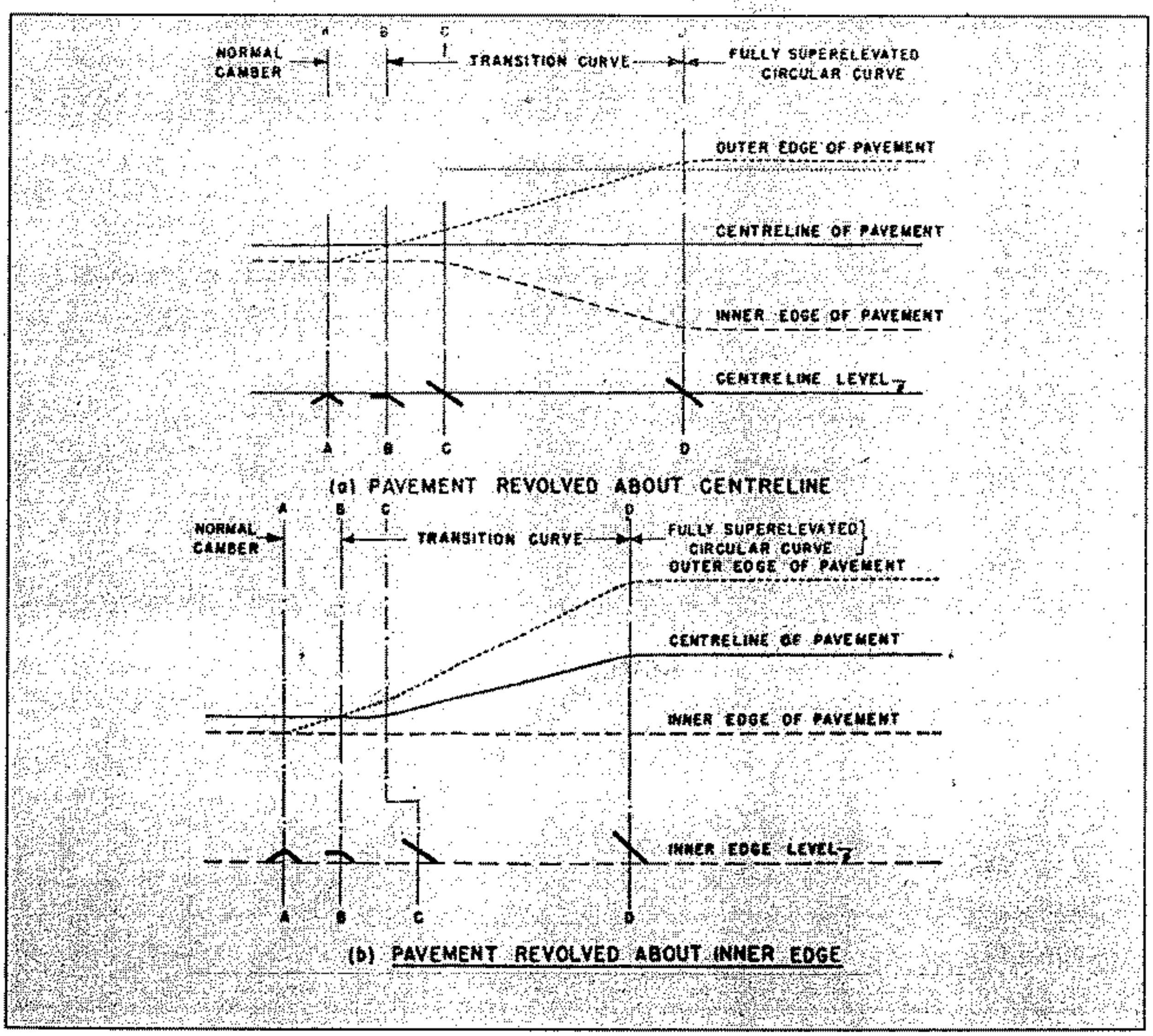
Method (1) rotating about centre line Method (2), rotating about the inner edge

- In this method as the pavement section is rotated about the centre line, the vertical profile of the centre line remains unchanged;
- The outer edge is banked and inner edge is depressed resulting in an advantage in balancing the earth work.

- The disadvantage of this method is the drainage problem due to depressing the inner edge below the general level.
- The drainage problem is of greater significance in areas with high rain fall when the subgrade is in cutting or in level terrain.
- If the subgrade is in embankment or when the road has a significant gradient to facilitate longitudinal drainage, there will be no drainage problem.

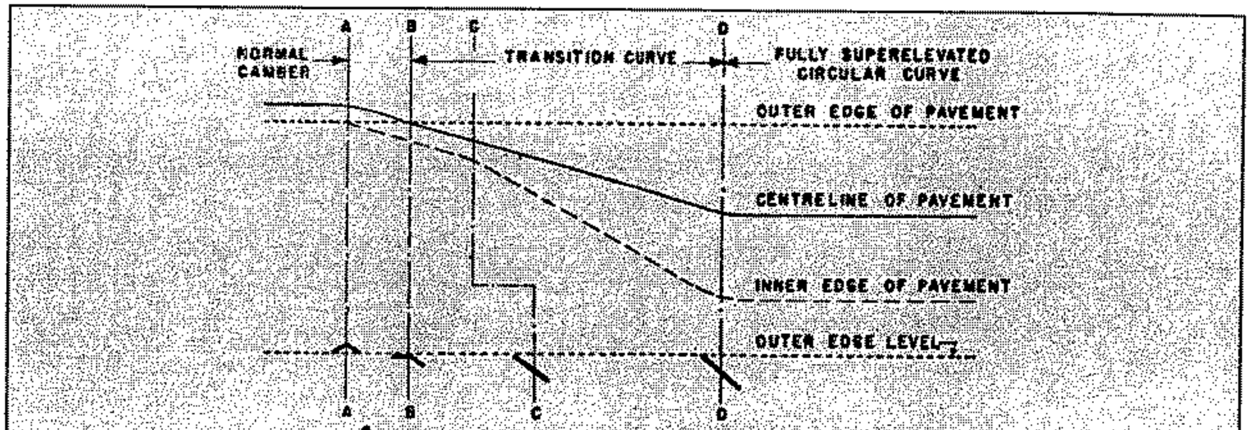
(b) rotation about the inner edge: Here the pavement is rotated raising the outer edge as well as the centre such that the outer edge is raised by the full amount of superelevation with respect to the inner edge.

- This method of rotating about the inner edge is preferable in very flat terrain in rain fall areas, when the road is not taken on embankment, in order to avoid the drainage problem.
- The entire pavement width and outer shoulder should be raised with respect to the inner edge by additional earth fill. In this case the centre of the pavement is also raised, which may be considered as a disadvantage of the method as the vertical alignment of the road is altered.
- The attainment of superelevation has been shown in detail in figure below.



(a) PAVEMENT REVOLVED ABOUT CENTRELINE

(b) PAVEMENT REVOLVED ABOUT INNER EDGE



(c) PAVEMENT REVOLVED ABOUT OUTER EDGE

Legend

Cross section at AA - normal camber
Cross section at BB - adverse camber removed
Cross section at CC - superelevation equal to camber
Cross section at DD - full superelevation achieved

Note: *The rate of change of superelevation should be minimum 1 in 150 for roads in plain and rolling terrain and 1 in 60 in mountainous and steep terrain. The actual rate used will determine the distances AB, BC and CD.*

- The outer half of the cambered section is raised to a horizontal position between A and B at the same rate of introduction of superelevation along the transition curve of length L_s. Thus at the tangent point B there is no negative superelevation.
- The superelevation should be attained gradually over the full length of transition curve so that the design superelevation is available at the starting point of the circular curve.
- In cases where transition curve cannot be provided for some reason, two-thirds of the superelevation may be attained at the straight portion before the start of the circular curve and the balance one-third in the circular curve.

Radius of Horizontal Curve

- The radius of the horizontal curve is an important design aspect of the geometric design. The maximum comfortable speed on a horizontal curve depends on the curve.
- Although it is possible to design the curve with maximum superelevation and coefficient of friction, it is not desirable because re-alignment would be required if the design speed is increased in future. Therefore, a ruling minimum radius R_{ruling} can be derived by assuming maximum superelevation and coefficient of friction. (e = 7% & μ = 0.15 normally unless given otherwise).

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

Ideally, the radius of the curve should be higher than R_{ruling}. IRC code gives the value of ruling minimum & absolute minimum value of radius of horizontal curve based on the ruling design speed & minimum design speed.

$$\boxed{R_{absmin} = \frac{(V_{min})^2}{g(e+\mu)}}$$

Radius of curve to be adopted should always be greater than ruling minimum. In most adverse case also the radius should not be less than absolute minimum.

Classification of road	Plain terrain		Rolling terrain		Mountainous terrain: Areas not affected by snow		Mountainous terrain: Snow bound areas		Steep terrain: Areas not affected by snow		Steep terrain: Snow bound areas	
	Ruling Minimum	Absolute Minimum	Ruling Minimum	Absolute Minimum	Ruling Minimum	Absolute Minimum	Ruling Minimum	Absolute Minimum	Ruling Minimum	Absolute Minimum	Ruling Minimum	Absolute Minimum
1. National Highways and State Highways	360	230	230	155	80	50	90	60	50	30	30	30
2. Major District Roads	230	155	155	90	50	30	60	33	30	14	33	15
3. Other District Roads	155	90	90	60	30	20	33	23	20	14	23	15
4. Village Roads	90	60	60	45	20	14	23	15	20	14	23	15

Example 9

A highway is provided with a horizontal curve of radius 300 m. in a certain locality. Calculate super-elevation required to maintain a design speed of 90 km/hr.

Calculate the maximum allowable speed if super elevation is limited to 0.07 and safe limit of transverse friction $f = 0.15$.

Note: For mixed traffic condition, the S.E should fully counteract the centrifugal force for 75% of design speed.

Sol. For a speed of (0.75 v), without considering transverse coefficient of friction

$$e = \frac{(0.278 \times 0.75V)^2}{(g \times R)} = \frac{(0.278 \times 0.75 \times 90)^2}{(9.81 \times 300)}$$

$$\therefore \quad e = 0.1195 > 0.07$$

Since this value is greater than 0.07, hence maximum value of S.E = 0.07 provided

if,
$$f = \frac{(0.278 \times 0.75 \times 90)^2}{gR} - 0.07$$

$$= \frac{(0.278 \times 0.75 \times 90)^2}{(9.81 \times 300)} - 0.07 = 0.142 < 0.15.$$

Therefore, maximum limit of S.E = 0.07 provided.

$\therefore$ Maximum allowable speed :

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

$$\therefore \quad v_{max} = \sqrt{127R(e+f)} = \sqrt{127 \times 300 \times (0.07 + 0.15)}$$

$$= 91.5 \text{ m/sec}$$

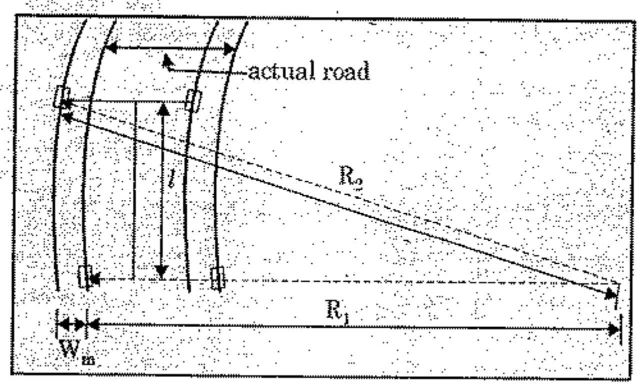
EXTRA WIDENING

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

Mechanical widening

- The reasons for the mechanical widening are : When a vehicle negotiates a horizontal curve, only front wheel can be turned, the rear wheels does not follow the same path as front wheel. It follow a path of shorter radius than the front wheels as shown in figure below.
- This phenomenon is called off-tracking, and has the effect of increasing the effective width of a road space required by the vehicle.
- In addition speeds higher than the design speed causes transverse skidding which requires additional width for safety purpose.
- Let R_1 is the radius of the outer track line of the rear wheel, R_2 is the radius of the outer track line of the front wheel l is the distance between the front and rear wheel, n is the number of lanes, then the mechanical widening W_m is derived below.

$$R_2^2 = R_1^2 + l^2$$
$$= (R_2 - W_m)^2 + l^2$$
$$= R_2^2 - 2R_2W_m + W_m^2 + l^2$$
$$2R_2W_m - W_m^2 = l^2$$
$$W_m(2R_2 - W_m) = l^2$$



Therefore the widening needed for a single lane road is: $W_m = \dfrac{l^2}{2R_2 - W_m}$

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

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

Hence, $$\boxed{W_m = \frac{nl^2}{2R}}$$

Psychological widening

- Widening of pavements has to be done for some psychological reasons also. There is a tendency for the drivers to drive close to the edges of the pavement on curves.

- Some extra space is to be provided for more clearance for the crossing and overtaking operations on curves. IRC proposed an empirical relation for the psychological widening at horizontal curves W_{ps}:

$$W_{ps} = \frac{v}{2.64\sqrt{R}} = \frac{V}{9.5\sqrt{R}}$$

Where W_{ps} is in m, v is in m/s, R is in m, V is in km/hr.

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

$$\boxed{W_e = W_m + W_{ps} = \frac{nl^2}{2R} + \frac{v}{2.64\sqrt{R}}}$$

- The above formula is applicable for two-lane or more
- Widening of single-lane roads however is somewhat different, since during crossing manouevers outer wheels of vehicles have in any case to use the shoulders whether on the straight or on the curve.
- It is therefore sufficient on single-lane roads if only the mechanical component of widening is taken into account.
- The extra width of carriageway to be provided at horizontal curves on single and two-lane roads as given in Table below.

Extra Width of Pavement at Horizontal Curves

Radius of curve (m)	Upto 20	21 to 40	41 to 60	61 to 100	101 to 300	Above 300
Extra width (m)						
Two-lane	1.5	1.5	1.2	0.9	0.6	Nil
Single-lane	0.9	0.6	0.6	Nil	Nil	Nil

Note: For R > 300 m, No extra widening is required

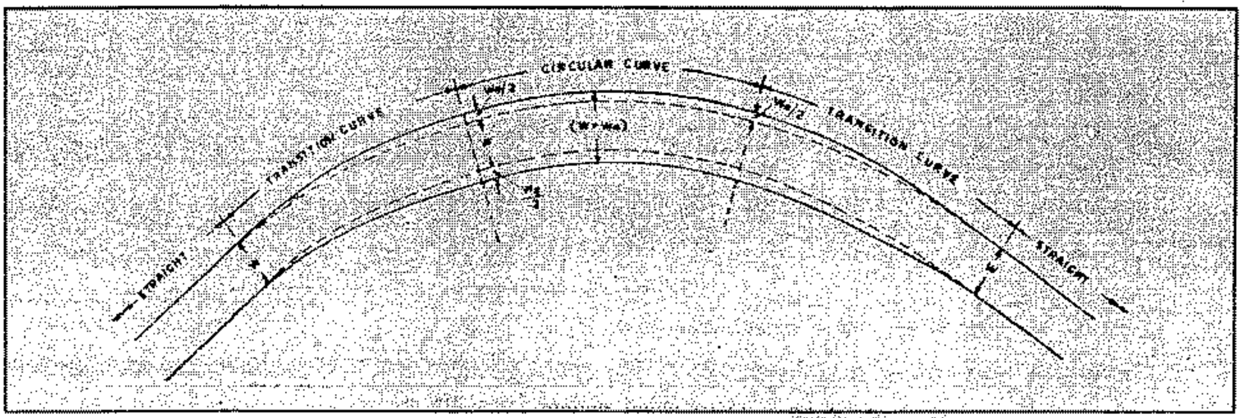
For multi-lane roads, the pavement widening may be calculated by adding half the widening for two-lane roads to each lane.

Methods of introducing extra widening

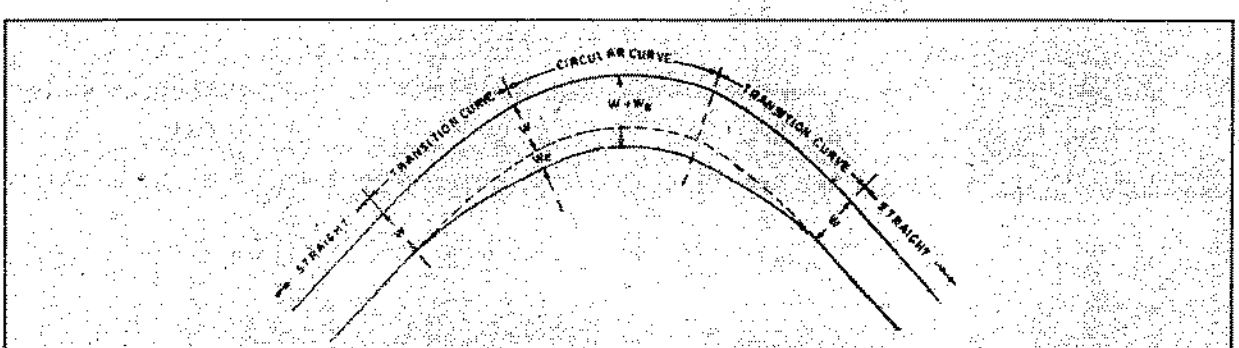
- The widening is introduced gradually, starting from the beginning of the transition curve or the tangent point (T.P.) and progressively increased at uniform rate, till the full value of designed widening 'W_e' is reached at the end of transition curve where full values of superelevation is also provided.
- The full value of extra width W_e is continued throughout the length of the circular curve and then decreased gradually along the length of transition curve.
- Usually the widening is equally distributed i.e., $W_e/2$ each on inner and outer sides of the curve.
- But on sharp curves of hill roads the extra widening W_e may be provided in full on inside of the curve.
- On horizontal circular curves without transition curves, two-thirds the widening is provided at the end of the straight section, i.e. before the start of the circular curve and the remaining one-

third widening is provided on the circular curve beyond the tangent point as in the case of superelevation.

- In such cases, the widening is provided on the inside of the curve.



Extra widening of pavement on horizontal curve



Widening of pavement on sharp curve

HORIZONTAL TRANSITION CURVES

- Transition curve is provided to change the horizontal alignment from straight to circular curve gradually and has a radius which decreases from infinity at the straight end (tangent point) to the desired radius of the circular curve at the other end (curve point).
- There are five objectives for providing transition curve and are given below:
  1. 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.
  2. To enable the driver turn the steering gradually for his own comfort and security.
  3. To provide gradual introduction of super elevation, and
  4. To provide gradual introduction of extra widening
  5. To enhance the aesthetic appearance of the road.

Type of transition curve

- Different types of transition curves are spiral or clothoid, cubic parabola, and Lemniscate.
- ***<u>IRC recommends spiral as the transition curve</u>*** because it fulfills the requirement of an ideal transition curve, that is:

(a) rate of change or centrifugal acceleration is consistent (smooth) and

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

Length of transition curve

The min length of the transition curve should be determined as the maximum of the following two criteria: rate of change of centrifugal acceleration, rate of change of superelevation

1.Rate of change of centrifugal acceleration

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

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

Therefore, the length of the transition curve L_{s1} in m is

$$\boxed{L_{s1} = \frac{v^3}{cR}}$$

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

$$\boxed{c = \frac{80}{75 + V}} \ m/s^3 \qquad [V = \text{velocity in km/hr}]$$

However, $c_{min} = 0.5$, $c_{max} = 0.8$

2. Rate of introduction of super-elevation

- The rate of change of superelevation, (i.e. the longitudinal grade developed at the pavement edge compared to through grade along centre line) should be such as not to cause discomfort to travellers or to make the road appear unsightly.
- The rate of change should not be steeper than 1 in 150 for road in plain & rolling terrain and 1 in 60 m mountainous/steep terrains

The formula for min. length of transition on this basis are

For plain & rolling terrain $L_{s2} = \frac{35v^2}{R} = \frac{2.7V^2}{R}$ [v is m/s, R is m]

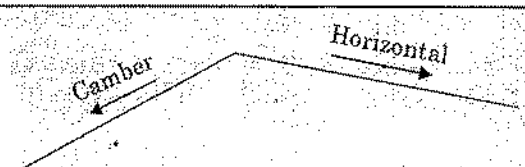
For steep & hilly terrain $L_{s2} = \frac{12.96v^2}{R} = \frac{V^2}{R}$ [V in km/hr, R is m]

Having regard to the above considerations, the minimum transition length for different speeds & curve radii are given in table.

Curve radius R (meters)	Plain and rolling terrain — Design speed (km/h) 100	80	65	50	40	35	Curve radius (metres)	Mountainous and steep (km/h) — Design speed (km/h) 50	40	30	25	20
	Transition length–metres							Transition length–metres				
45					NA	70	14				NA	30
60				NA	75	55	20				35	20
90				75	50	40	25			NA	25	20
100			NA	70	45	35	30			30	25	15
150			80	45	30	25	40		NA	25	20	15
170			70	40	25	20	50		40	20	15	15
200		NA	60	35	25	20	55		40	20	15	15
240		90	50	30	20	NR	70	NA	30	15	15	15
300	NA	75	40	25	NR		80	55	25	15	15	NR
360	130	60	35	20			90	45	25	15	15	
400	115	55	30	20			100	45	20	15	15	
500	95	45	25	NR			125	35	15	15	NR	
600	80	35	20				150	30	15	15		
700	70	35	20				170	25	15	NR		
800	60	30	NR				200	20	15			
900	55	30					250	15	15			
1000	50	30					300	15	NR			
1200	40	NR					400	15				
1500	35						500	NR				
1800	30											
2000	NR											

Some times, however, we find out the length of superelevation as follows. [from Rate of introduction of superelevation]

- At the beginning of the transition curve, one end of the camber is brought to horizontal.



- Now the horizontal leg is rotated about centre line to achieve a superelevation equal to camber. If after this stage further superelevation is achieved by rotating both leg about centre. Then in the total length of transition curve (L_s) the outer edge is raised by $\frac{eB}{2}$ with respect to central line.

Where, e = superelevation (final value) ; B = width of pavement

If widening is provided B will also gradually increase with superelevation.

Thus, raising of outer edge with respect to centre line will be $\frac{e(W + W_e)}{2}$

Where e = actual width on straight reach

W_e = Widening of pavement.

Rate of application of superelevation $= \dfrac{e(W + W_e)}{2 \times L_s}$

Where, L_s = Length of transition curve.

As per IRC code, rate for plain terrain $= \dfrac{1}{N} = \dfrac{1}{150}$

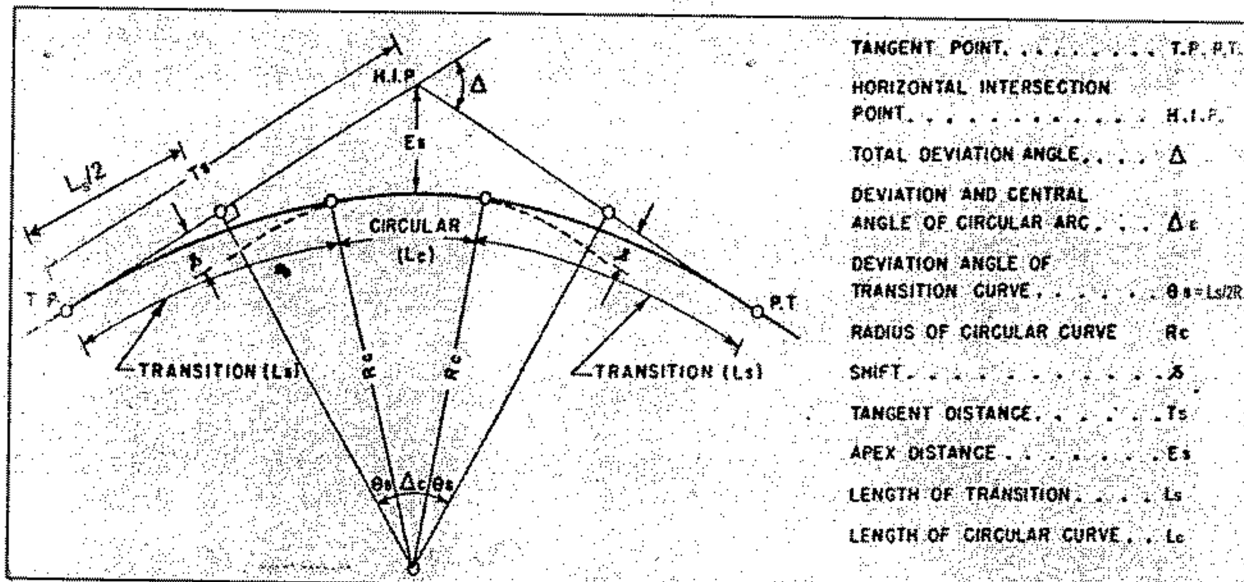
$$\Rightarrow \quad \frac{e(W + W_e)}{2 \times L_s} = \frac{1}{N}$$

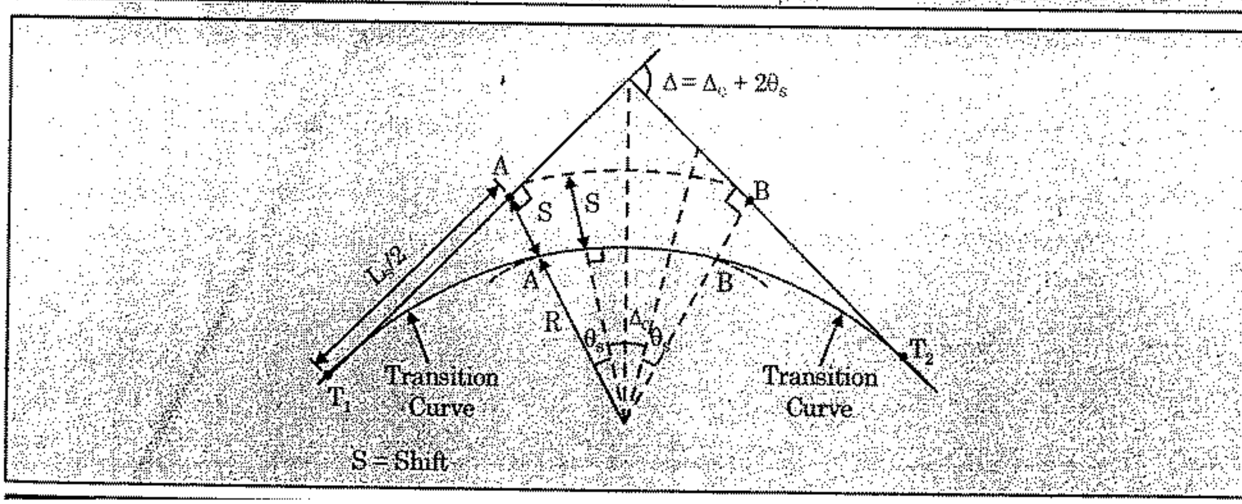
$$\Rightarrow \quad \boxed{L_s = \frac{e(W + W_e)N}{2}}$$

If however, the pavement is rotated about inner edge then length of Transition Cuve is given by $\boxed{L_S = eN(W + W_e).}$

Setting out of Transition curve

The various elements of transition circular curve combination are as shown below





When transition curved is introduced between a straight & circular curve, the circular curve has to be shifted so that the transition curve meets the circular curve tangentially. The shift is given by

$$\boxed{\text{Shift, } S = \frac{L_s^2}{24R}}$$

$$\boxed{\text{Total Length of Curve} = LC + 2LS}$$

L_C = Length of circular curve AB
L_S = Length of transition curve = $T_1A = BT_2$

$$\theta_S = \frac{L_C}{2R}$$

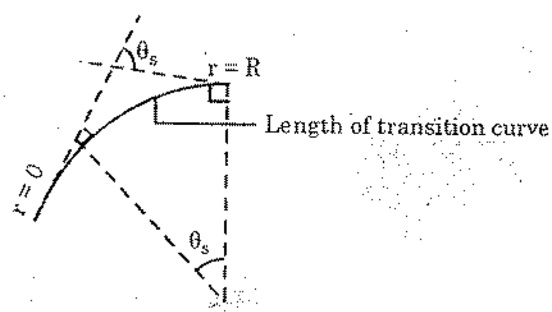
$$\Delta = \Delta_C + 2\theta_S$$

$$\Delta_C = \Delta - 2\theta_S = \Delta - \frac{L_C}{2R}$$

$$\boxed{L_C = \frac{2\pi R}{2\pi}\Delta_C}, \text{ where } \Delta_C \text{ is in radian}$$

$$\boxed{\text{Tangent Distance} = (R+S)\tan\frac{\Delta}{2} + \frac{L_S}{2}}$$

Note :



If L_S = Length of transition curve, then $\theta_S = \dfrac{L_S}{2R}$

Setting out of curve is done according to either off-set method or by polar deflection angle method as discussed in surveying book.

Example 10

An Expressway four lane divided, passing through a flat terrain has a horizontal curve of radius equal to Ruling minimum radius. If the design speed is 120km/hr. Calculate the following elements of the curve.
(i) Ruling minimum radius. (ii) Super Elevation (iii) Extra widening (iv) Length of transition curve. Suggest the best suitable shape of the transition curve.

Sol. Data given,

Design speed, V = 120km/hr

As per IRC, maximum allowable super elevation, e = 0.07

Design coefficient of lateral friction, f = 0.15

The Ruling minimum radius, $R_{ruling} = \dfrac{(0.278V)^2}{g(e+f)} = \dfrac{(0.278\times120)^2}{9.81\times(0.07+0.15)} = 515.66\text{m}.$

(ii) The super elevation for 75% design speed is calculated neglecting the lateral friction

$$e = \frac{(0.75 \times 0.278V)^2}{gR} = \frac{(0.75 \times 0.278 \times 120)^2}{(9.81 \times 515.66)}$$

$$= 0.124 > 0.07$$

Provide maximum S.E = 0.07, if $f = \left(\frac{(0.278 \times V)^2}{gR} - e\right)$

$$= \frac{(0.278 \times 120)^2}{9.81 \times 515.66} - 0.07 = 0.149 < 0.15$$

Therefore provide maximum S.E. = 0.07

(iii) The Extra widening, $E_W = \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}} = \frac{4 \times (6.1)^2}{2 \times 515.66} + \frac{120}{9.5\sqrt{515.66}} = 0.701$

(iv) Length of transition curve

(a) As per Rate of change of Radial Acceleration

$$C = \frac{v^2}{R \times T}$$

$$\Rightarrow \quad T = \frac{v^2}{R \times C}$$

$$\Rightarrow \quad \frac{L}{v} = \frac{v^2}{(R \times C)}$$

$$L = \left(\frac{v^3}{R \times C}\right)$$

As per IRC where $C = \frac{80}{(75+V)}$, where V is in km/hr and $0.5 \le C \le 0.8$.

$$\therefore \quad C = \frac{80}{(75+120)} = 0.41$$

$$\therefore \quad L = \frac{(0.278 \times 120)^3}{(515.66 \times 0.41)} = 175.60\text{m} \quad \text{...(i)}$$

(b) As per IRC, the length of transition curve for a flat terrain is given by

$$L_s = \frac{2.7V^2}{R} = \frac{2.7 \times (120)^2}{515.66} = 75.39 \text{ m.} \quad \text{...(ii)}$$

From (i) & (ii), the length of transition curve will be maximum of (a) and (b)

i.e $L_s = 175.60.$

Spiral transition curve is the most ideal choice for highways.

Example 11

Find the length of transition curve and extra width of pavement required on a horizontal curve of radius 300m of a 2-lane highway passing through rolling terrain for a design speed of 80 km/hr. Assume all other data as per IRC Recommendation.

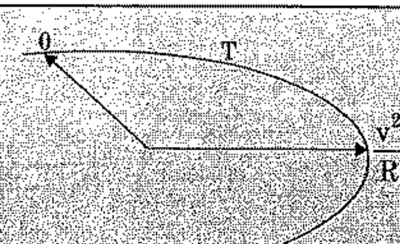
Sol. Data given,

Radius of curve, R = 300m, No. of lanes = 2

Design speed, V = 80 km/hr

Length of transition curve.

(a) As per rate of change of radial acceleration



$$C = \frac{v^2}{R \times T}$$

$$\Rightarrow \quad T = \frac{v^2}{(R \times C)}$$

$$\Rightarrow \quad \frac{L}{v} = \frac{v^2}{(R \times C)}$$

$$\therefore \quad L = \frac{v^3}{(R \times C)}$$

where,
$$C = \frac{80}{(75+V)} = \frac{80}{(75+80)} = 0.516$$

$$L = \frac{(0.278 \times V)^3}{R \times C} = \frac{(0.278 \times 80)^3}{(0.516 \times 300)} = 71.06\text{m} \quad \ldots \text{(i)}$$

(b) As per IRC, the length of Horizontal transition curve is given by for plain and rolling terrain.

$$L = \left(\frac{2.7V^2}{R}\right) = \frac{2.7 \times (80)^2}{300} = 57.6 \text{ m} \quad \ldots \text{(ii)}$$

Hence the length of transition curve provided, will be the maximum of (a) and (b)

Therefore, $L_s = 71.06\text{m}.$

(ii) Extra widening,
$$E_w = \left(\frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}}\right)$$

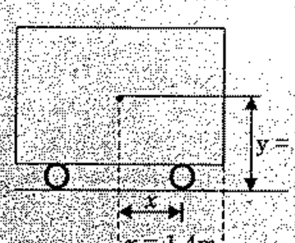
As per IRC, the length of wheel base, $l = 6.1$ m.

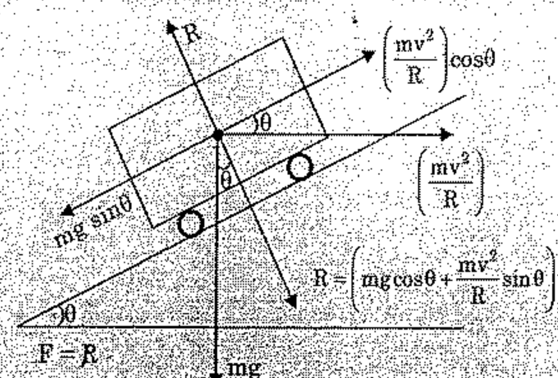
$$\therefore \quad E_w = \frac{2 \times (6.1)^2}{2 \times 300} + \frac{80}{9.5\sqrt{300}} = 0.610 \text{ m.}$$

Example 13

A truck with C. G. at $x = 1.4$m and $y = 1.7$m. is travelling on curved road of radius 200m and S.E. = 0.05. Determine the minimum safe speed to avoid both slipping and overturning assuming coefficient of side friction = 0.15. Sketch, explain and derive the expression, for condition when overturning is critical.

Sol.





To avoid slipping:

Total force in outer radial direction should be ≤ total resisting force in inward direction along the road surface $\frac{mv^2}{R}\cos\theta \le (mg\sin\theta + F)$

$$\frac{mv^2}{R}\cos\theta = mg\sin\theta + f(mg\cos\theta + \frac{mv^2}{R}\sin\theta)$$

$$\Rightarrow \left(\frac{v^2}{R}\right) = g\tan\theta + fg + f\frac{v^2}{R}\tan\theta$$

$$\Rightarrow \frac{v^2}{R}(1 - ef) = g(f + e) \qquad [\because e = \tan\theta]$$

$$\Rightarrow \boxed{\frac{v^2}{gR} = \frac{(e+f)}{(1-ef)}} \qquad \ldots(\text{i})$$

Condition to avoid slipping, given data : $\left.\begin{matrix} e = 0.05 \\ f = 0.15 \end{matrix}\right\} R = 200\text{m}.$

$$\Rightarrow \frac{(0.278v)^2}{9.81 \times R} = \left(\frac{e+f}{1-ef}\right)$$

$$\Rightarrow v = \sqrt{\frac{9.81}{(0.278)^2}R\left(\frac{e+f}{1-ef}\right)} = \sqrt{127R\left(\frac{e+f}{1-ef}\right)}$$

$$= \sqrt{127 \times 200\left(\frac{0.05+0.15}{1-.05 \times 0.15}\right)} = 71.54 \text{ km/hr}$$

(2) To avoid overturning

Overturning moment about outer wheel should be ≤ the total resisting moment.

$\Rightarrow$ Overturning moment = Resisting moment

$$\Rightarrow \frac{mv^2}{R}\cos\theta \times y = (mg\sin\theta \times y) + \left(mg\cos\theta + \frac{mv^2}{R}\sin\theta\right) \times x$$

$$\Rightarrow \frac{v^2}{R}y = (g\tan\theta) \times y + gx + \frac{v^2}{R}\tan\theta \times x$$

$$\Rightarrow \frac{v^2}{R}y = gey + gx + \frac{v^2}{R}ex \qquad (\because e = \tan\theta)$$

$$\Rightarrow \frac{v^2}{R}(y - ex) = g(ey + x)$$

$$\Rightarrow \frac{v^2}{gR} = \frac{(ey+x)}{(y-ex)}$$

$$v = \sqrt{127R\left(\frac{ey+x}{y-ex}\right)} = \sqrt{127 \times 200\left(\frac{0.05 \times 1.7 + 1.4}{1.7 - 1.4 \times 0.05}\right)}$$

$$v = 152.12 \text{ km/hr.}$$

Example 14

Calculate the values of ruling minimum and absolute minimum radius of horizontal curve of national highway in plain terrain. Assume ruling design speed and minimum design speed values as 100 and 80 km/hr respectively.

Sol. Data given,

Ruling design speed = 100 km/hr

Minimum design speed = 80 km/hr

$\therefore$ As per IRC, e = 0.07; f = 0.15

$$\therefore \quad \text{Ruling minimum radius} = \frac{V^2}{127(e+f)} = \frac{(100)^2}{127\times(0.07+0.15)} = 357.9 \text{ m.}$$

The absolute minimum radius is calculated from the minimum design speed v = 80 km/hr.

$$R_{min} = \frac{(80)^2}{127\times(e+f)} = \frac{(80)^2}{127\times(0.07+0.15)} = 229.1 \text{ m.}$$

Example 15

A national highway passing through a flat terrain has a horizontal curve of radius equal to the Ruling minimum radius. If the design speed is 100 km/hr. Calculate absolute minimum sight distance, superelevation, extra widening and length of transition curve. Assume necessary data suitably.

Sol. Data Given,

Design Speed = 100 km/hr

As per IRC, for absolute minimum radius, e = 0.07, f = 0.15

$$\therefore \quad R_{min} = \frac{(0.278V)^2}{g(e+f)} = \frac{(0.278\times100)^2}{9.81(0.07+0.15)} = 358.095\text{m}$$

We know that, Sight Distance S.D. = (Lag distance + Braking distance)

$$= 0.278Vt_R + \frac{(0.278V)^2}{2g(f\pm n\%)} = 332.103 \text{ m}$$

[n = 0 because the curve is passing through flat terrain]

S.E. : Neglecting the lateral coefficient of friction, e is designed for 75% of design speed.,

$$e = \frac{(0.75v)^2}{gR} = \frac{(0.75\times0.278\times100)^2}{(9.81\times358.095)} = 0.123 > 0.07$$

$$f = \frac{V^2}{gR} - 0.07$$

$$= 0.1496$$

since, f is less than 0.15, we will provide e = 0.07

Hence, provide Max. S.E., e = 0.07

Extra Widening $$E_w = \left(\frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}}\right)$$

$$E_w = \frac{2\times(6.1)^2}{2\times(358.095)} + \frac{100}{9.5\sqrt{358.095}} = 0.660$$

$\therefore$ Assume two lane highway.

Therefore total width of pavement = 7.0 + 0.66 = 7.66m.

Length of transition curve

(i) As per rate of change of radial acceleration

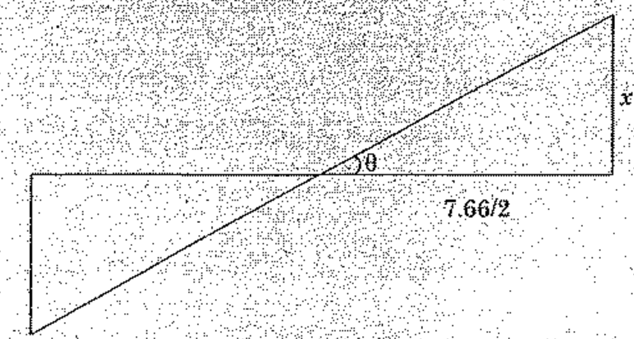
$$L_s = \frac{v^3}{(R \times c)} \qquad \left[\text{where } c = \frac{80}{(75+V)} = 0.457\right]$$

$$= \frac{(0.278 \times 100)^3}{(358.095 \times 0.457)} = 131.29 \text{ m} \qquad \text{(i)}$$

(ii) As per IRC method, for flat terrain

$$L_s = \frac{2.7V^2}{R} = \frac{2.7 \times (100)^2}{(358.095)} = 75.398 \text{ m} \qquad \text{...(ii)}$$

(iii) As per introduction of S.E., $L = 150x$



$$x = \left(\frac{7.66}{2}\right)\tan\theta = \left(\frac{7.66}{2}\right) \times 0.07 = 0.2681$$

$$\therefore \quad L = 150x = 40.215 \text{ m} \qquad \text{...(iii)}$$

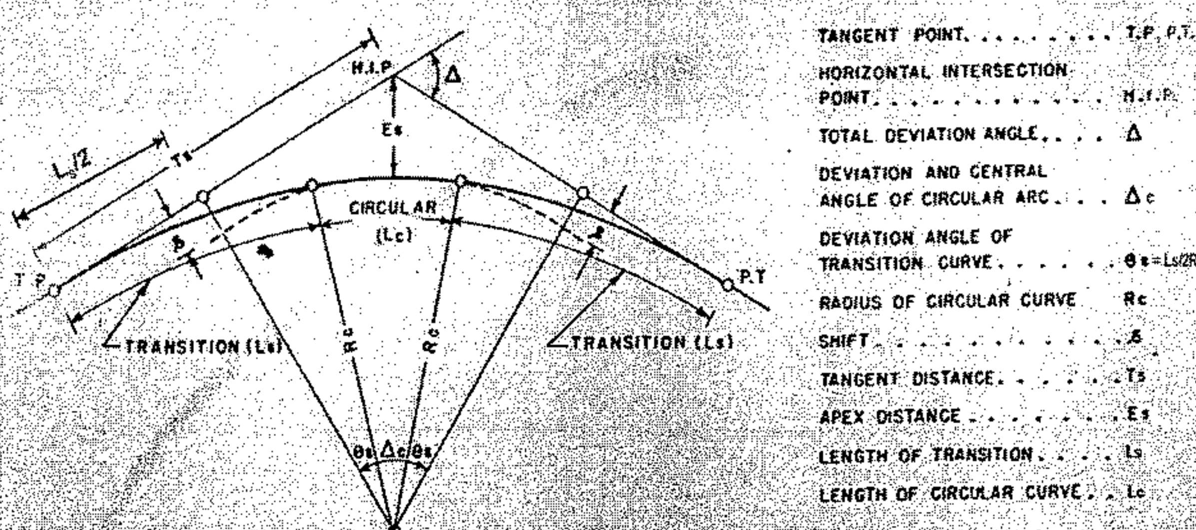
From (i), (ii) & (iii), higher will be the length of transition curve

$$\therefore \quad L_s = 131.29 \text{ m} \approx 132 \text{ m}$$

Example 16

A two lane pavement (7.0) m on a national highway in hilly terrain (snow bound) has a curve of radius 60m. The design speed is 40 km/hr. Determine the length of the transition curve, Determine the total length of the curve and tangent length if the deflection angle is 60°. Make suitable assumptions.

Sol.



Data given,
$R = 60.00$ m
$V = 40$ km/hr

where, $\theta_s = \left(\frac{L_s}{2R_C}\right)$ radians $\rightarrow$ spiral angle

$$S = \left(\frac{L_s^2}{24R_c}\right) = \text{Shift}$$

where, Δ_c = Angle of the circular curves; L_s = Spiral length; T_s = Tangent length.

Assumption:

(i) $e = 0.07$

(ii) Rate of attainment of S.E = 1 in 60.

Length of transition curve:

(i) As per rate of change of radial acceleration .

$$L = \left(\frac{v^3}{Rc}\right)$$

where, $$C = \left(\frac{80}{75+V}\right) = 0.696 \qquad 0.5 \le C \le 0.8$$

$$L = \frac{(0.278\times40)^3}{(60\times0.696)} = 32.93 \qquad \text{... (I)}$$

$$e = \frac{(0.75v)^2}{gR} = \frac{(0.75\times0.278\times40)^2}{(9.81\times60)} = 0.12 > 0.07.$$

Therefore provide maximum S.E, $e = 0.07$

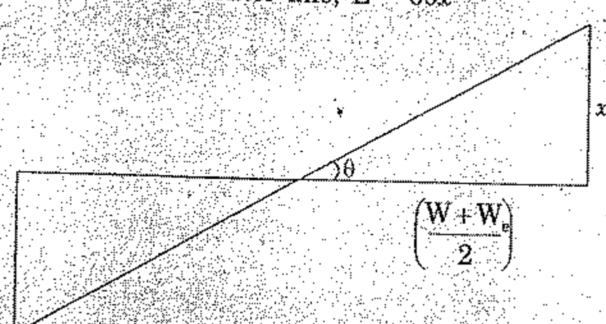
$$E_w = \text{Extra Widening} = \left(\frac{nl^2}{2R}+\frac{V}{9.5\sqrt{R}}\right) = \frac{2\times(6.1)^2}{2\times60}+\frac{40}{9.5\sqrt{60}}$$

$$= 1.16 \simeq 1.2\text{m}.$$

Total Pavement width = $(W + E_W) = 7 + 1.2 = 8.2$m

(ii) Length of transition curve as per rate of introduction of S.E.

(a) If pavement is rotated about center line, $L = 60x$



where, $$x = \left(\frac{W+W_e}{2}\right)\tan\theta = \frac{8.2}{2}\times0.07 = 0.287$$

$$L = 60 \times 0.287 = 17.22 \text{ m} \qquad \text{...(II)}$$

(iii) As per IRC for hilly terrain,

$$L = \left(\frac{V^2}{R}\right) = \frac{(40)^2}{60} = 26.67\text{m} \quad \text{... (III)}$$

From (I), (II) & (III), maximum length will be the length of transition cure

Length of transition curve = 32.93 m

Total deflection angle = 60° = Δ

$$\text{Shift S} = \left(\frac{L_s^2}{24R}\right) = \frac{(32.93)^2}{(24\times 60)} = 0.75\text{m}$$

$$\text{Tangent Length} = \left(\frac{L_S}{2}\right) + (R_C + S)\tan\left(\frac{\Delta}{2}\right)$$

$$= \frac{32.93}{2} + (60 + 0.75)\tan\left(\frac{60°}{2}\right) = 51.54\text{ m}$$

$$\text{Spiral angle, } \theta_s = \frac{L_s}{2R_C}\text{ radian}$$

$$= \left(\frac{32.8}{2\times 60}\right)\times\left(\frac{180}{\pi}\right) = 15.66°$$

$$\Delta_C = (\Delta - 2\theta_S) = (60 - 2 \times 15.66) = 28.68°$$

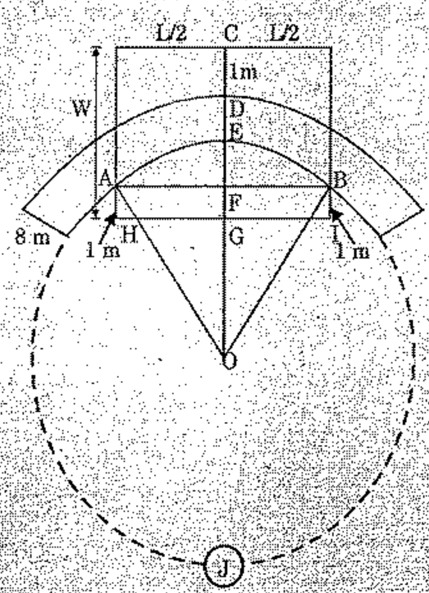
$$\therefore \quad \text{Length of circular curve} = \left(\frac{2\pi R}{360}\right)\times \Delta_C = \frac{2\pi\times 60}{360}\times 28.68 = 30.04\text{ m}$$

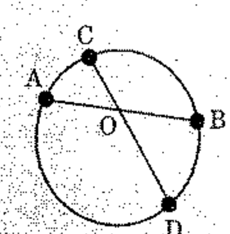
Total length of curve = 30.04 + 2 × 32.8 = 95.64 m

Example 17

A single rectangular bridge span of length L and width W is used on a horizontal curve. If the roadway is 8m wide and minimum. Clearance of 1m is desired between the edge of pavement and bridge rail. Show that the min radius of curvature $R = \dfrac{L^2}{8(W-10)} + \left(\dfrac{W-10}{2}\right)$

Sol.





Property of circle is that OA × OB = OC × OD

The bridge shall be provided as shown in figure. Where minimum clearance between bridge rail and Edge of pavement 1m is available at centre of outer edge and ends of inner edge.

From the figure $CD = 1m;\ FG = AH = BI = 1m$

$EF = W - (8 + 1 + 1) = (W - 10)$

$\therefore$ From property of circle, $AF \times FB = (EF \times FJ)$

$$\left(\frac{L}{2}\right)\times\left(\frac{L}{2}\right) = (W - 10) \times [2R - (W - 10)]$$

$$\frac{L^2}{4} = 2R(W - 10) - (W - 10)^2$$

$$2R\,(W - 10) = (W - 10)^2 + \frac{L^2}{4} \Rightarrow R = \frac{(W-10)^2}{2(W-10)} + \frac{L^2}{8(W-10)}$$

$$R = \frac{L^2}{8(W-10)} + \frac{(W-10)}{2}$$

SETBACK DISTANCE

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

  (1) sight distance (SSD, ISD and OSD), (2) radius of the curve, (3) length of the curve.

  There are two cases in the calculation of setback distance

  (1) when length of curve is greater than the stopping distance.

  (2) when length of curve is smaller than the stopping distance.

Case-1 When length of curve is greater than the stopping distance (single lane road) i.e., $L_c > S$

Sight distsance is measured along centre line.

$$\frac{\alpha}{360} = \frac{S}{2\pi R} \quad (\alpha \text{ in degree})$$

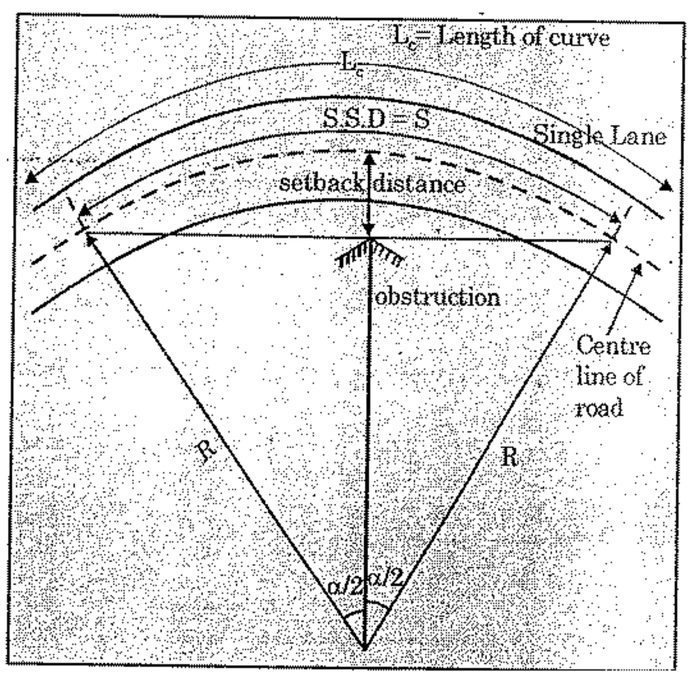
$$\alpha = \frac{360.S}{2\pi R}$$

$$\frac{\alpha}{2} = \frac{360.S}{2\times 2\pi R}$$

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

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

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



- Minimum stopping distance should correspond to the stopping sight distance

  Hence S should correspond to stopping sight distance to find out minimum setback distance.

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

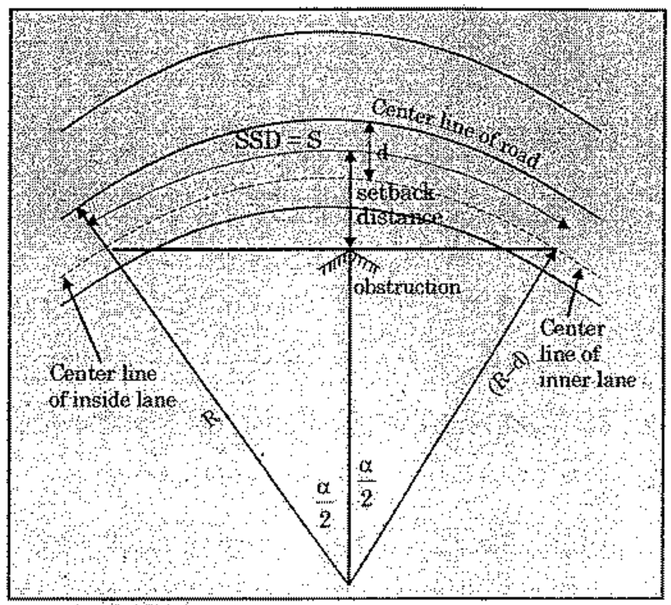
$$\frac{\alpha}{360} = \frac{S}{2\pi(R-d)}$$

(α in degree)

$$\frac{\alpha}{2} = \frac{360.S}{2\times 2\pi(R-d)}$$

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

$$\boxed{\text{Setback distance} = R-(R-d)\cos\left(\frac{\alpha}{2}\right)}$$



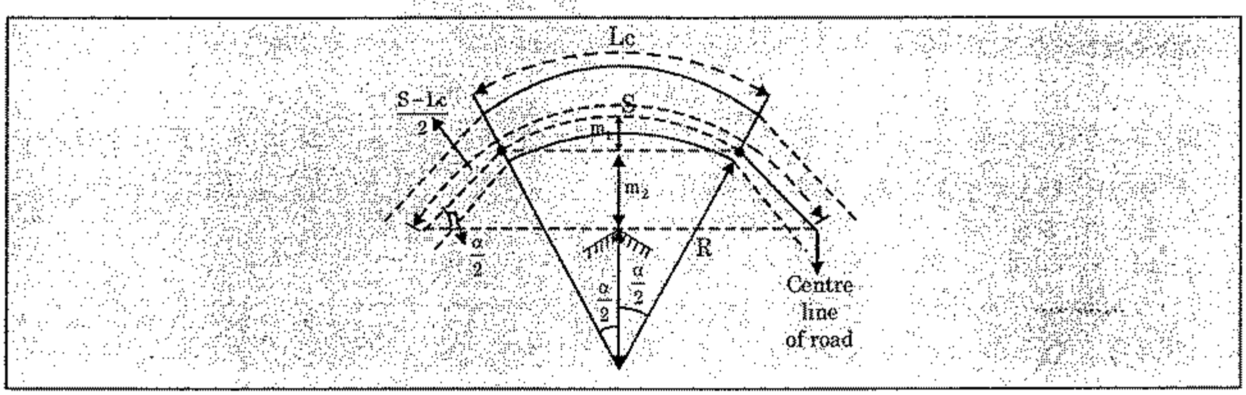
where R = Radius of curve or Radius of centre line of multilane road.

d = distance between centre line of road and the centre line of inner lane

S is the stopping distance measured along the centre line of inner lane.

Case (2) $L_c < S$

For single lane:



$$m_1 = R - R\cos\left(\frac{\alpha}{2}\right)$$

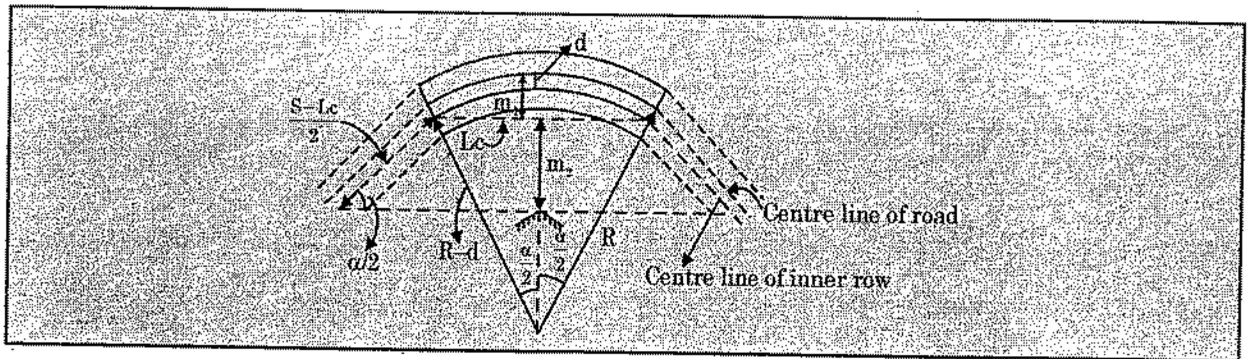
$$m_2 = \frac{(S-L_c)}{2}\sin\left(\frac{\alpha}{2}\right)$$

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

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

where, $$\frac{\alpha}{2} = \frac{180L_c}{2\pi(R)},$$

For multilane road



$$m_1 = R - (R - d)\cos\frac{\alpha}{2}$$

$$m_2 = \frac{S-Lc}{2}\sin\frac{\alpha}{2}$$

$$\text{Set back distance} = m_1 + m_2$$

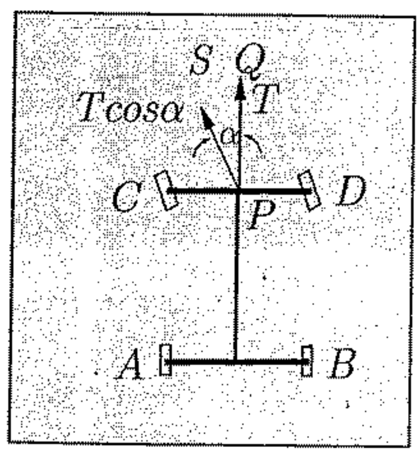
$$\boxed{\text{where, } \frac{\alpha}{2} = \frac{180L_c}{2\pi(R-d)}}$$

Note : Actually Lc should be measured along the centre line ofroad. However the difference in value in setback distance will not be significant if L_C is taken along the centre line of inner row.

Clearance of obstruction upto setback distance is important.

CURVE RESISTANCE

When the vehicle negotiates a horizontal curve, the direction of rotation of the front and the r ear wheels are different.

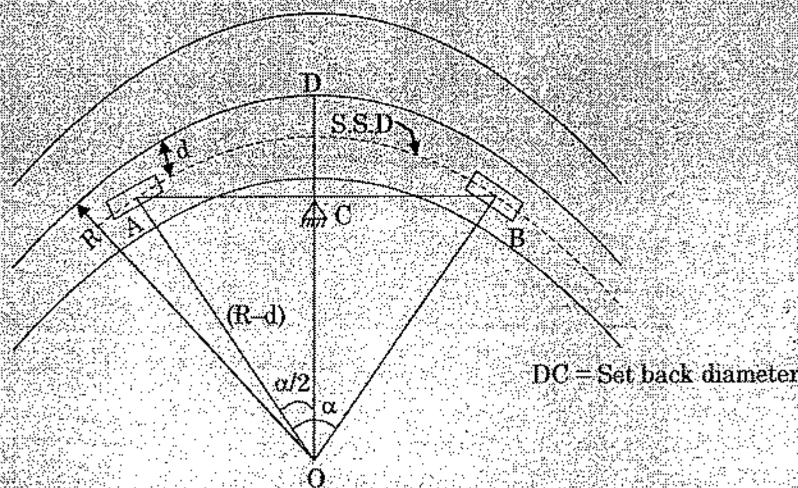


- The front wheels are turned to move the vehicle along the curve, whereas the rear wheels seldom turn. This is shown in figure below. If the vehicle is driven by rearwheel, the rear wheels exert a tractive force T in the PQ direction.
- The tractive force available on the front wheels is T cosα in the PS direction as shown in the figure.
- This is less than the actual tractive force, T applied. Hence, the loss of tractive force for a vehicle to negotiate a horizontal curve is: CR = T – T cosα = curve resistance.
- This problem doesnot exist in front wheel driven vehicle.
- Most of the commercial vehicles are rear wheel driven. Hence additional tractive force is required to negotiate the curve.

Example 18

There is a horizontal highway curve of radius 400m. and length 200m on this highway. Compute the set-back distances required from the center line on the inner side of the curve so as to provide for (a) SSD of 90m. (b) Safe SSD of 300 m.
The distance between the center lines of the road and the inner lane is 1.9 m.

Sol.



(a) Given data,

S.S.D = 90; Length of circular curve = 200m; R = 400 m; d = 1.9 m.

(i) L_c > S.S.D

$$\text{Set-Back distance } m = CD = (OD - OC)$$

$$= R - (R - d)\cos\left(\frac{\alpha}{2}\right) \quad \text{...(i)}$$

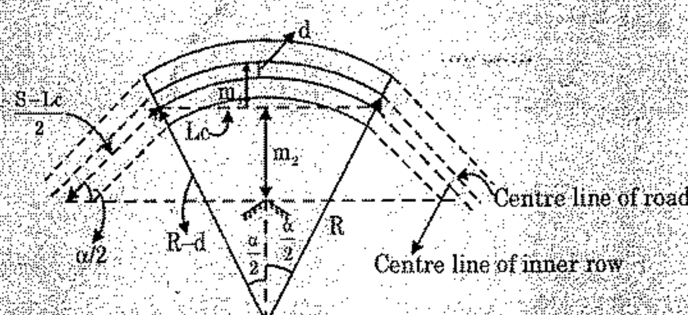
From Figure,

$$\left(\frac{\alpha}{360}\right) = \frac{S.S.D}{2\pi(R-d)}$$

$$\frac{\alpha}{2} = \frac{180\times 90}{2\pi\times(400-1.9)} = (6.48)^\circ$$

$$m = 400 - (400 - 1.9)\cos(6.48)^\circ = 4.44 \text{ m}$$

(b) L_c < S.S.D



Given Data, S.S.D = 300m.; Lc = 200m

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

$$\frac{\alpha}{360} = \frac{L_c}{2\pi(R-d)}$$

$$\left(\frac{\alpha}{2}\right) = \frac{180L_c}{2\pi(R-d)} = \frac{180\times200}{2\pi\times(400-1.9)} = 14.39°$$

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

$$= 400-(400-1.9)\cos(14.39)+\frac{(300-200)}{2}\sin(14.39°)$$

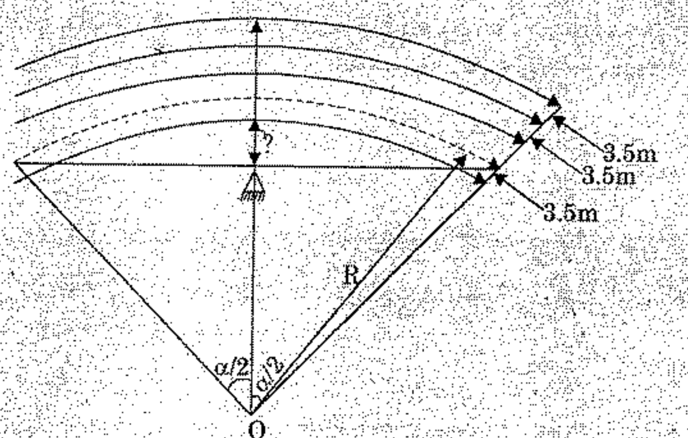
$$= 14.39 + 12.43 = 26.81 \text{ m.}$$

∴ Minimum Set-Back distance required from the centre line of the roads on the inner side of the povement to provide on OSD of 300m = 27m.

Example 19

A six-lane divided carriageway has a curve 1000m long and a radius of 500 m. The safe stopping sight distance is 200m. Calculate the minimum set-back distance from the inner edge of the road to the edge of a building to ensure safe visibility. The pavement width per lane is 3.5m.

Sol.



Data given,

Length of curve = 1000 m.; R = 500 m.; S.S.D = 200 m

$$\frac{\alpha}{360} = \frac{S.S.D}{2\pi\left[R-(3.5\times2+\frac{3.5}{2})\right]}$$

$$\Rightarrow \quad \frac{\alpha}{2} = \frac{180\times200}{2\pi\left[500-\left(3.5\times2+\frac{3.5}{2}\right)\right]} = 11.66 \text{ degree.}$$

$$\text{Set-back distance} = R-\left(R-3.5\times2-\frac{3.5}{2}\right)\cos 11.66°$$

$$= 500-(500-8.75)\cos 11.66°$$

$$= 18.89\text{m (from centre line)}$$

From figure distance from inner edge to obstruction

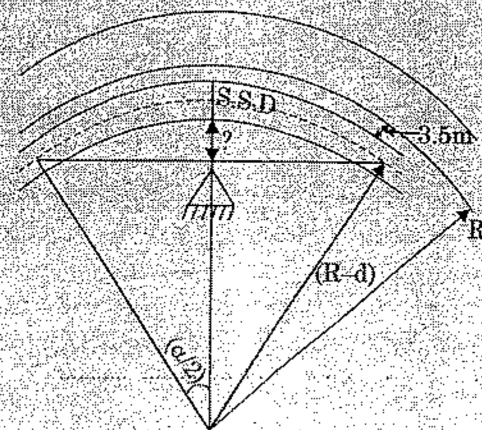
$$= (18.89 - 3.5\times3) = 8.39 \text{ m.}$$

Example 20

Calculate the set-back distance from the inner edge of the curve in a four lane divided carriageway, the length being 1500m. The stopping sight distance 250m. The lane width being 3.5m and the radius of the curve being 400 m.

Sol. Data given,

L = 1500; S.S.D = 250m; R = 400m



$$\frac{\alpha}{360} = \frac{S.S.D}{2\pi(R-d)}$$

$$\left(\frac{\alpha}{2}\right) = \frac{(250\times180)}{2\pi\left[400-\left(3.5+\frac{3.5}{2}\right)\right]} = 18.143°$$

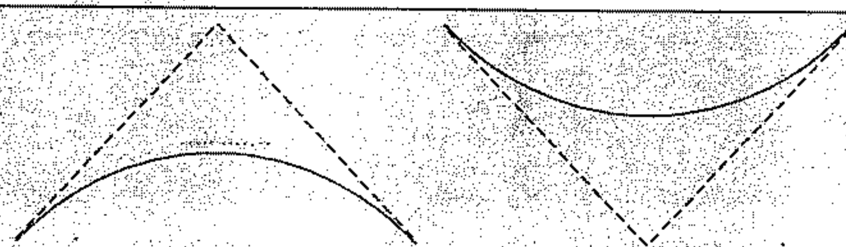
Set-back distance, $m = R - (R - d)\cos\left(\frac{\alpha}{2}\right)$

$$= 400-\left(400-3.5-\frac{3.5}{2}\right)\cos(18.143°) = 24.876 \text{ m (from center line)}$$

$\therefore$ Distance from inner edge to obstruction = (27.876 – 7.00) = 17.87 m.

VERTICAL ALIGNMENT

- The vertical alignment of a road consists of gradients (straight lines in a vertical plane) and vertical curves.
- The vertical alignment is usually drawn as a profile, which is a graph with elevation as vertical axis and the horizontal distance along the centre line of the road as the horizontal axis.
- Just as a circular curve is used to connect horizontal straight stretches of road, vertical curves connect two gradients.



Vertical Summit Curve   Vertical Valley Curve

- When these two curves meet, they form either convex or concave.
- The former is called as summit curve, while the latter is called a valley curve.

Gradient

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

Effect of gradient

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

Representation of gradient

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

IRC Specifications for gradients for different roads (in percentage)

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

Types of gradient

(1) Ruling gradient
(2) Limiting gradient
(3) Exceptional gradient
(4) 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.
- The ruling gradient is adopted by the designer by considering a particular speed as the design speed and for a design vehicle with standard dimensions.
- With the maximum pulling power, the vehicle would be able to sustain the same speed even on long sections only up to a certain gradient. This is when the maximum power developed by the engine is equal to the power required to overcome the resistances to motion on the grade at this speed.
- Therefore this gradient, is the one which should be adopted as a ruling gradient by the designer for this vehicle and the design speed.

Limiting gradient

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

Exceptional gradient

- Exceptional gradient are very steeper gradients given at unavoidable situations. They should be limited for short stretches not exceeding about 100 metres at a stretch. In mountainous and steep terrain, successive exceptional gradients must be separated by a minimum 100 metre length gentler gradient. At hairpin bends, the gradient is restricted to 2.5%. The rise of elevation over a length of 2 km shall not exceed 100 m in mountaneous terrain & 120 m in steep terrain.
- IRC code has given following recommendation for the gradients.

Gradients for Roads in different Terrains

S.No.	Terrain	Ruling gradient	Limiting gradient	Exceptional gradient
1.	Plain or rolling	3.3 percent (1 in 30)	5 percent (1 in 20)	6.7 percent (1 in 15)
2.	Mountainous terrain, and steep terrain having elevation more than 3,000 m above the mean sea level	5 percent (1 in 20)	6 percent (1 in 16.7)	7 percent (1 in 14.3)
3.	Steep terrain upto 3,000 m height above mean sea level	6 percent (1 in 16.7)	7 percent (1 in 14.3)	8 percent (1 in 12.5)

Critical length of the grade

The maximum length of the ascending gradient which a loaded truck can operate without undue reduction in speed is called critical length of the grade. A reduction in speed of 25 kmph is a reasonable value.

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. If the terrain is flat, then for flatter- gradient drain depth will become much deeper than the normal level of ground. To avoid it, minimum gradient is provided for drainage purpose and it depends on the rain fall, type of soil and other site conditions. A minimum of 1 in 500 as minimum gradient of road may be sufficient for concrete drain and 1 in 200 for open soil drains are found to give satisfactory performance.

GRADE COMPENSATION

- While a vehicle is negotiating a horizontal curve, if there is a gradient also, then there will be increased resistance to traction due to both curve and the gradient.

- 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 the curve.
- This reduction in gradient at the horizontal curve is called grade compensation, which is intended to off-set the extra tractive effort involved at the curve.
  1. Grade compensation is not required for grades flatter than 4% because the loss of tractive force is negligible.
  2. Grade compensation is $\frac{30+R}{R}\%$, where R is the radius of the horizontal curve in meters.
  3. The maximum grade compensation is limited to $\frac{75}{R}\%$
  4. The gradient need not be reduced beyond 4%

Example 21

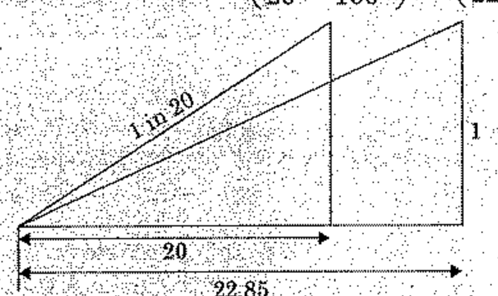
If Ruling gradient is 1 in 20. What will be the grade compensation and compensated gradient for a curve of radius 120m.

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

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

$$= \frac{(30+120)}{120}\%\text{ to}\left(\frac{75}{120}\right)\% = 1.25\% \text{ to } 0.625\%$$

$$\text{Compensated gradient} = \left(\frac{1}{20}-\frac{0.625}{100}\right) = \left(\frac{1}{22.85}\right)$$



Example 22

A national highway passing through a mountainous terrain has a horizontal curve of radius equal to ruling minimum radius.

Design all geometrical features of the curve. Calculate set back distance for ISD also. It is a two lane highway.

Sol. As per IRC, for national highway,

Ruling design speed = 50 km/hr for mountainous region and e = 0.10 and f = 0.15.

(i) Ruling main radius, $R_{min} = \frac{(0.278\times V)^2}{g(e+f)} = \frac{(0.278\times 50)^2}{9.81\times(0.10+0.15)}$

$= 78.78\text{m} = 79\text{m}.$

(ii) For S.E. Neglecting the lateral friction coeffn superelevation should fully counteract the centrifugal force for 75% of design speed.

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

Therefore provide maximum S.E. for mountainous region e = 0.10

If, $f < 0.15$

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

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

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

(iv) Length of transition curve

(a) As per rate of change of radial acceleration

$$L = \frac{v^3}{(R \times c)} \quad \text{where } c = \frac{80}{(75 + V)} = \frac{80}{(75 + 50)} = 0.64$$

V is in km/hr ; $0.5 < c < 0.8$

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

(b) As per IRC, for mountainous region,

$$\text{Length of transition curve, } L = \left(\frac{V^2}{R}\right)$$

$$L = \frac{(50)^2}{79} = 31.64 \text{ m.} \quad \text{...(ii)}$$

From (i) & (ii), maximum length will be length of transition curve.

Therefore, provide length of transition curve = 53.11m = 54m.

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

For R = 79m, Grade compensation $= \left(\frac{75}{79}\right)\% = 0.95\% = 0.0095$

If Gradient is 1 in 20

$$\text{Max. Gradient} = \left(\frac{1}{20} - .0095\right) = \frac{1}{24.7}$$

(vi) Set Back Distance:

Set-Back distance is measured from center of road.

Assume L_c > I.S.D

I.S.D. = 2 × (S.S.D)

For S.S.D. without considering gradient

$$S.S.D = 0.278Vt_R + \frac{(0.278 \times V)^2}{2g \times (f \pm n\%)}$$

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

$$\text{I.S.D.} = 2 \times (\text{S.S.D}) = 2 \times 62.9 = 125.77 \text{ m}.$$

$$\frac{\alpha}{360} = \frac{\text{ISD}}{2\pi(R-d)}$$

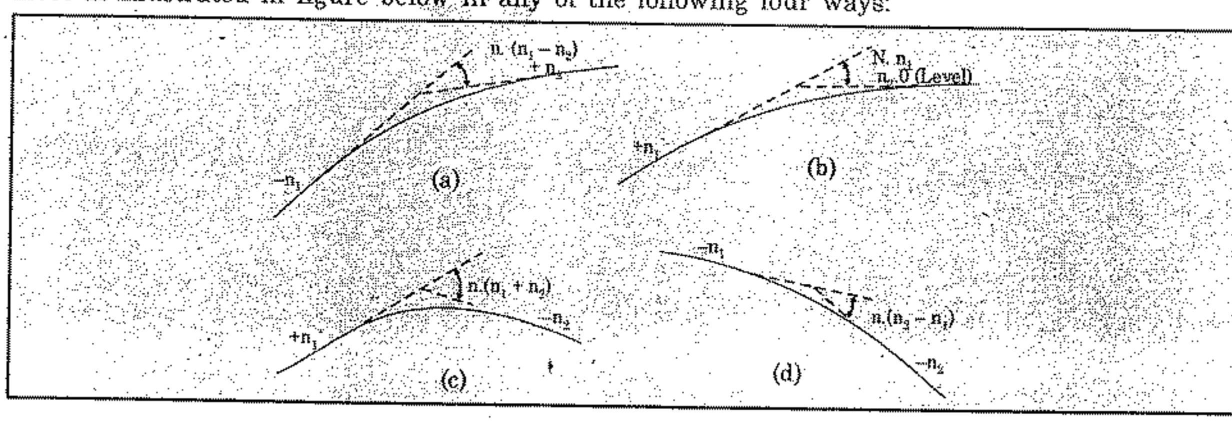
$$\frac{\alpha}{2} = \frac{180\text{ISD}}{2\pi \times (R-d)} = \frac{180 \times 125.77}{2\pi \times (79 - \frac{8.05}{4})} = (46.80)° \ldots (i)$$

$$\text{Set Back Distance, ED} = (\text{OD} - \text{OE}) = R - (R-d)\cos\left(\frac{\alpha}{2}\right)$$

$$= 79 - (79 - 2.01)\cos(46.80)° = 26.297 \text{ m.} = 26.3 \text{ m.}$$

SUMMIT CURVE

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



Summit curve

1. when a positive gradient meets another positive gradient
2. when positive gradient meets a flat gradient
3. when an ascending gradient meets a descending gradient
4. when a descending gradient meets another descending gradient

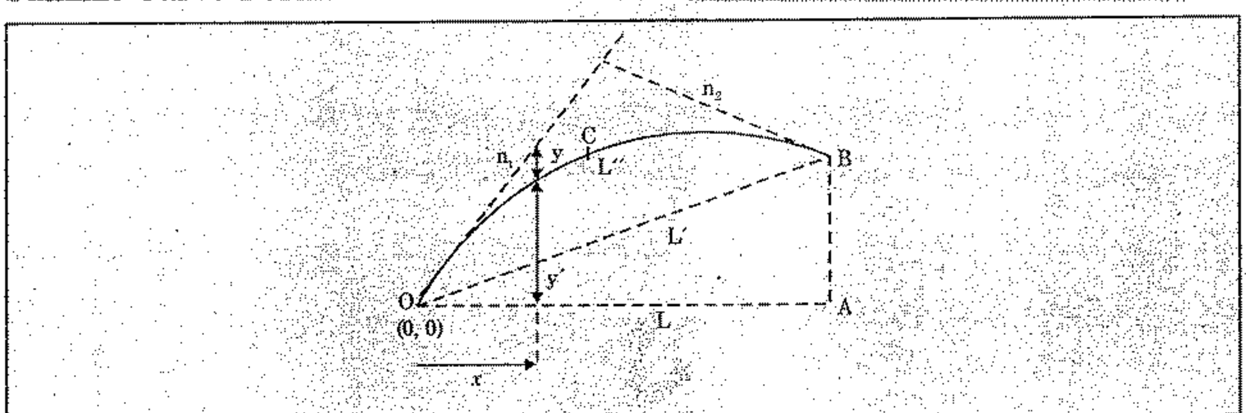
- The design of a summit cune is governed by consideration of sight distance. Unless the summit is so low as not to interfere with visibility.
- The dynamics of movement over an ordinary summit curve is of little consequence.

- This can be inferred from two considerations : (1) The centrifugal force generated by the movement of the vehicle along the curve acts practically in opposition to the force of gravity and is, therefore, beneficial in so far as it relieves the pressure on the tyres and springs of the vehicle; (2) Vertical deviation angles on roads are so small because the summit curves prescribed by the sight distance are so long and easy that "shock" is automatically rendered imperceptible to the travellers.
- Many curve forms can be used with satisfactory results, the common practice has been to used parabolic curves in summit curves.
- This is primarily because of the ease with it can be laid out as well as allowing a comfortable transition from one gradient to another.
- Although a circular curve offers equal sight distance at every point on the curve, for very small deviation angles a circular curve and parabolic curves are almost congruent.
- Furthermore, the use of parabolic curves were found to give excellent riding comfort.

Design consideration

- In determining the type and length of the vertical curve, sight distance requirements for the safety is most important on summit curves.
- The stopping sight distance or absolute minimum sight distance should be provided on these curves and where overtaking is not prohibited. overtaking sight distance or intermediate sight distance should be provided as far as possible.

Summit Curve Formula



The simple parabolic equation of the summit curve is given by

$$y' = ax^2 + bx + c$$

For small deviation angle, we do not differentiate between OA(i.e L), OB(ie L′) adn OCB (ie L″).

$$\frac{dy'}{dx} = 2ax + b$$

at $$x = 0,\ \frac{dy}{dx} = b = n_1$$

at $$x = L,\ \frac{dy}{dx} = n_2$$

$$\Rightarrow \quad 2aL + b = n_2$$

$$\Rightarrow \quad a = \frac{n_2 - n_1}{2L}$$

at $x = 0$, $y = 0 \Rightarrow c = 0$

$$\Rightarrow \quad y' = \frac{n_2 - n_1}{2L}x^2 + n_1 x$$

$$y = n_1 x - y'$$

$$= n_1 x - \frac{n_2 - n_1}{2L}x^2 - n_1 x$$

$$\Rightarrow \quad \boxed{y = \frac{n_1 - n_2}{2L}x^2}$$

If

$$N = n_1 - n_2 \begin{cases} \text{where } n_1 \,\&\, n_2 \text{ are used with proper sign} \\ \text{i.e. upward gradient is taken as } (+) \,\&\, \text{downward} \\ \text{gradient is taken as } (-) \end{cases}$$

$$y = \frac{N}{2L}x^2$$

Radius of Curvature of Summit Curve

We know that curvature is given by

$$\text{Curvature} = \frac{\frac{d^2y}{dx^2}}{\left[1 + \left(\frac{dy}{dx}\right)^2\right]^{3/2}}$$

Vertical curves are generally flat hence curvature canbe writen as

$$\text{Curvature} = \frac{1}{R} = \frac{d^2y}{dx^2} = \frac{N}{L}$$

$$\Rightarrow \quad \boxed{\text{Radius of Curvature} = \frac{L}{N}}, \; L = \text{Length of curve}; \; N = n_1 - n_2$$

Heighest Point on Summit Curve

It is sometimes important to know the position d of the highest point on a vertical curve for the purpose of layout of drainage appurtenances and for ascertaining vertical clearances in restricted locations as road under bridges, etc.

From the derivation for eq. of summit curve

$$y' = \frac{-(n_1 - n_2)}{2L}x^2 + n_1 x \quad \ldots\ldots\ldots$$

$$\text{For highest Point} = \frac{dy'}{dx} = 0$$

$$\Rightarrow \quad \frac{-(n_1 - n_2)}{L}x + n_1 = 0$$

$$\Rightarrow \quad x = \frac{n_1 L}{n_1 - n_2} = \frac{n_1 L}{N}$$

$$\boxed{\text{The highest point is given by, } x = \frac{n_1 L}{N}}$$

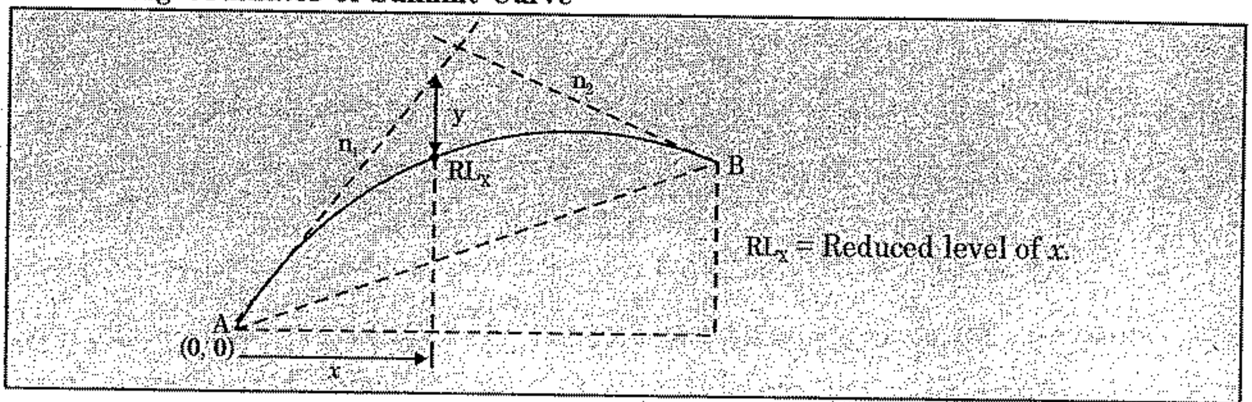
n_1 = 1st gradient

L = Length of curve

N = $(n_1 - n_2)$, n_1 & n_2 with this algebric sign is used

Note : When the two grades are equal the curve would be symmetrical about the vertical bisector of the intersecting angle and the highest point would also lie on this bisector. When the two grade are unequal the curve would be tilted and the highest point of the curve would line on the side of the flatter gradient.

Calculating Ordinates of Summit Curve



RL of a point at a distance x.

$$RL_x = (\text{RL. of A}) + n_1x - y$$

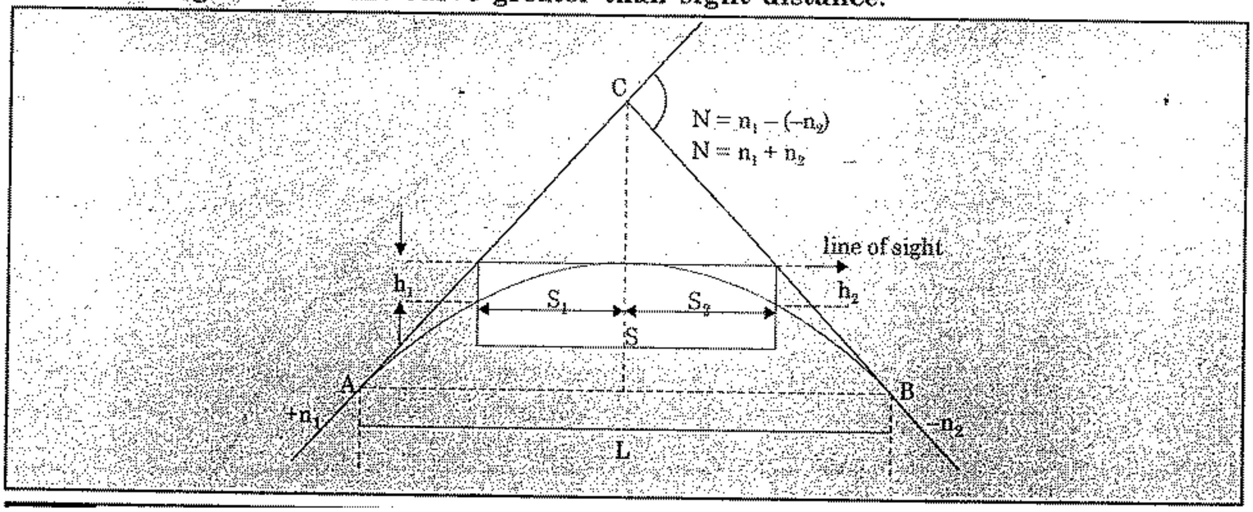
$$RL_x = (RL)_A + n_1x - \frac{Nx^2}{2L}$$

$$\Rightarrow \quad RL_B = (RL)_A + n_1L - \frac{NL^2}{2L}$$

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.
- The length of the curve is guided by the sight distance consideration.
- Equation of the parabola is given by $y = ax^2$, where $a = \frac{N}{2L}$, where N is the deviation angle and L is the length of curve. While deriving the length of the curve, two situations can arise ie. (i) length of the curve is greater than the sight distance and (ii) length of the curve is less than sight distance.
- Let L is the length of the summit curve, S is the SSD/ISD/OSD, N is the deviation angle, h_1 driver's eye height (1.2m), and h_2 the height of the obstruction, then the length of the summit curve can be derived for the following two cases.

Case A : Length of summit curve greater than sight distance.



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

S = Stopping distance (N could be SSD, OSD or ISD)

Case B : Length of summit curve less than sight distance

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

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

Hence, for SSD,

$$L = \frac{NS^2}{4.4}, \text{ for } L > SSD$$

$$L = 2S - \frac{4.4}{N}, \text{ for } L < SSD$$

Similarly for OSD,

$$L = \frac{NS^2}{9.6}, \text{ for } L > OSD/ISD$$

$$L = 2S - \frac{9.6}{N}, \text{ for } L < OSD/ISD$$

- When the deviation angle is small, the length of summit curve generally works out less than the sight distance. In very small deviation angles, the length required some times works out as a negative value indicating that there is no problem of sight restriction at the summit curve. But for comfort in driving and to avoid shock, it is necessary to introduce a vertical curve except perhaps in very flat grades. The minimum length of the curve should be as indicated in Table below. This Table also shows the maximum grade change not requiring a vertical curve.

Minimum Length of vertical curves

Design Speed in km/hr	Maximum grade change (percent) not requiring a curve	Minimum length of vertical curve (m)
Upto 35	1.5	15
40	1.2	20
50	1.0	30
65	0.8	40
80	0.6	50
100	0.5	60

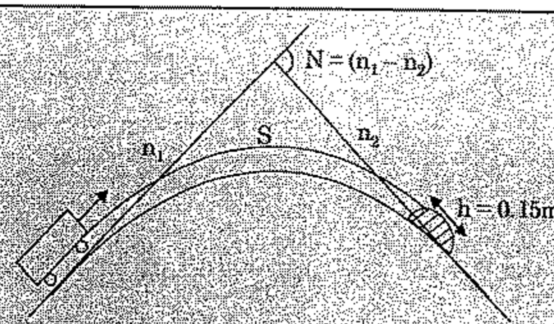
Example 23

An ascending gradient 1 in 60 meets a descending gradient of 1 in 50. Find the length of vertical summit curve for a stopping sight distance of 180m.

Sol. Data given,

S.S.D = 180 m.

$n_1 = \frac{1}{60}$, $n_2 = -\frac{1}{50}$



$$N = (n_1 - n_2)$$

$$= \frac{1}{60} - \left(\frac{-1}{50}\right) = \frac{1}{60} + \frac{1}{50} = \left(\frac{11}{300}\right)$$

∴ The length of summit vertical curve is given by

$$L = \frac{NS^2}{\left(\sqrt{2H} + \sqrt{2h}\right)^2}, \text{ where } L > S.S.D.$$

Where, L = Length of summit curve in m.

S = Stopping sight distance in m.

H = Height of eye level of driver above roadway surface in m.

h = Height of object above the pavement surface in m.

$N = (n_1 - n_2)$

As per IRC, H = 1.2m.

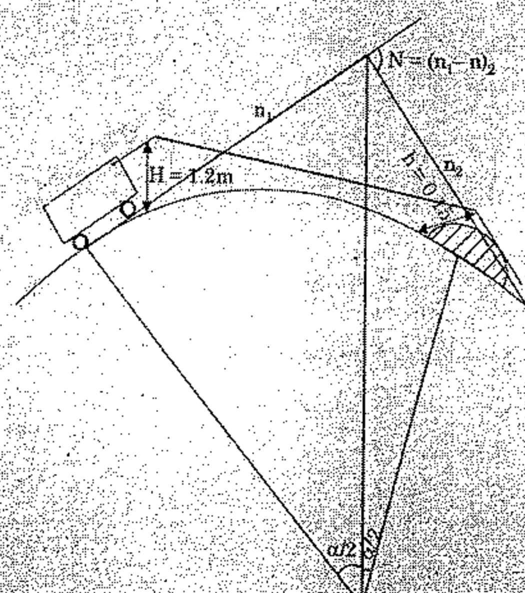
h = 0.15m.

$$L = \frac{\left(\frac{11}{300}\right) \times (180)^2}{\left(\sqrt{2 \times 1.2} + \sqrt{2 \times 0.15}\right)^2} = 270.18 \text{ m} > 180 \text{ m}.$$

Example 24

Calculate the length of summit curve for a stopping sight distance of 180m. On a national highway at the junction of an upward gradient of 1 in 200 and a downward gradient of 1 in 200. Assume the height of the driver to be 1.2 m. and height of object above roadway to be 0.15m.

Sol.



Data given,

S.S.D = 180 m; $n_1 = \left(\frac{1}{200}\right)$; $n_2 = \left(\frac{-1}{200}\right)$; H = 1.2m; h = 0.15m.

Assume that L > S.S.D

Therefore we know that, $$L = \frac{NS^2}{\left(\sqrt{2H}+\sqrt{2h}\right)^2}$$

$$\because \quad N = (n_1 - n_2) = \left[\frac{1}{200} - \left(\frac{-1}{200}\right)\right] = \frac{2}{200} = \left(\frac{1}{100}\right)$$

$$\therefore \quad L = \frac{NS^2}{\left(\sqrt{2H}+\sqrt{2h}\right)^2} = \frac{1\times(180)^2}{100\left(\sqrt{2\times1.2}+\sqrt{2\times0.15}\right)^2}$$

$$= 73.64\text{m} < \text{S.S.D}$$

Therefore, $$L = \left(2S - \frac{4.4}{N}\right) = \left(2\times180 - \frac{4.4}{\left(\frac{1}{100}\right)}\right) = -80.00 \text{ m}$$

This condition is also not fulfilled.

The conclusion is that the grade change is too small and does not need a vertical curve for stopping sight distance. However, for aesthetic purpose provide a minimum length of 60 m

Example 25

Design a summit curve for a National Highway for a stopping sight distance of 180m at the junction of a rising gradient of 1 in 50 and a falling gradient of 1 in 30. Set out the curve with a chord 25m long. Determine the R.L of the point immediately below the intersection point of the grade lines and also the R.L of the highest point on the curve.

Sol. Data Given,

S.S.D = 180m; $n_1 = \frac{1}{50}$; $n_2 = -\left(\frac{1}{30}\right)$

Assume L > S.S.D,

$$\therefore \quad N = \frac{1}{50} - \left(\frac{-1}{30}\right) = \frac{4}{75} = 0.053$$

Therefore, $$L = \frac{NS^2}{\left(\sqrt{2H}+\sqrt{2h}\right)^2} = \left(\frac{NS^2}{4.4}\right) = \frac{4\times(180)^2}{(75\times4.4)} = 392.73\text{m}$$

[$\because$ H = 1.2m, h = 0.15m]

Hence our assumption is correct. Provide a curve 400 m long

$$\therefore \quad \text{Equation of parabola} = y = \left(\frac{Nx^2}{2L}\right) = \left(\frac{0.053x^2}{2\times400}\right) = \left(\frac{0.053x^2}{800}\right)$$

The summit point in at a distance of

$$\left(\frac{N_1L}{N}\right) = \left(\frac{0.02\times400}{0.053}\right) = 150.94\text{m}$$

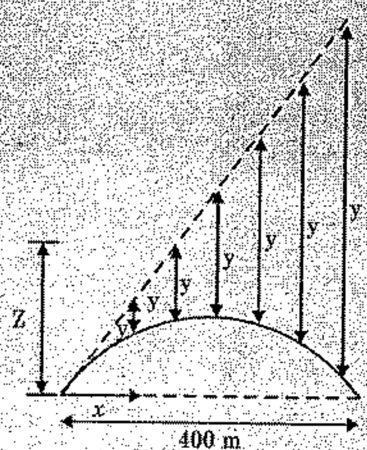
$$\therefore \quad y = \frac{0.053\times(150.94)^2}{800} = 1.509\text{m}$$

Assume RL of started of curve = 10 m

R.L. of Summit = (10.00 + 150.94 × 0.02 – 1.509) = 11.51m

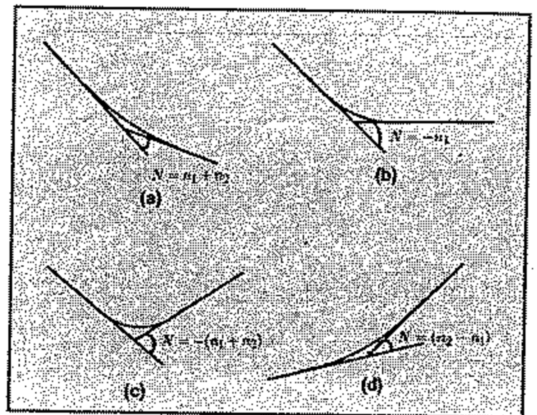
The vertical Distance between the point of vertical intersection and the curve

$$= \left(\frac{NL}{8}\right) = \frac{(0.053 \times 400)}{8} = 2.650\text{m}$$



Station	x(m)	y(m)	Z=Difference between start of curve and grade line N_1 (i.e. N_1x)	R.L. $=RL_A+Z-Y$
Start of curve				
1	0	0.00	0.00	10.00
2	25	0.041	0.500	10.459
3	50	0.166	1.00	10.834
4	75	0.372	1.50	11.128
5	100	0.662	2.00	11.338
6	125	1.035	2.50	11.465
7	150	1.490	3.00	11.510
8	175	2.029	3.50	11.471
9	200	2.650	4.00	11.350
10	225	3.351	4.50	11.149
11	250	4.140	5.00	10.860
12	275	5.010	5.50	10.490
13	300	5.962	6.00	10.038
14	325	6.998	6.50	9.502
15	350	8.116	7.00	8.884
16	375	9.316	7.5	8.184
17	400	10.600	8.00	7.400

VALLEY CURVE



Valley curve or sag curves are vertical curves with convexity downwards. 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 flat gradient
3. When a descending gradient meets an ascending gradient
4. When an ascending gradient meets another ascending gradient.

Design considerations

- There is no restriction to sight distance at valley curves during day time.
- But visibility is reduced during night. In the absence or inadequacy of street light, the only source for visibility is with the help of headlights.
- Hence valley curves are designed taking into account of headlight distance.
- In valley curves, the centrifugal force will be acting downwards along with the weight of the vehicle, and hence imapct to the vehicle will be more. This will result in jerking of the vehicle and cause discomfort to the passengers.
- Thus the most important design factors considered in valley curves are (1) impact-free movement of vehicles at design speed and (2) availability of stopping sight distance under headlight of vehicles for night driving.
- For gradually introducing and increasing the centrifugal force acting downwards, the best shape that could be given for a valley curve is a transition curve. Cubic parabola is generally preferred in vertical valley curves. However if deviation angle is small the path traversed by spiral, leminscate or cubic parabola are all same.
- During night, under headlight driving condition, head light sight distance should be at least equal to the stopping sight distance. There is no problem of overtaking sight distance at night since the other vehicles with headlights could be seen from a considerable distance.

Length of the valley curve

- The valley curve is made fully transitional by providing two similar transition curves of equal length the transitional curve is set out by a cubic parabola $y = bx^3$ where $b = \frac{2N}{3L^2}$. However for small deviation angles we generally use parabolic curve the design. The length of the valley transition curve is designed based on two criteria:

1. ***Comfort criteria***: In this criteria, allowable rate of change of centrifugal acceleration is limited to a comfortable level of about $0.6 m/sec^3$.
2. ***Safety criteria***: In this criteria, the driver should have adequate headlight sight distance at any part of the country.

Comfort criteria

The length of the valley curve based on the rate of change of centrifugal acceleration that will ensure comfort: Let c is the rate of change of acceleration, R the minimum radius of the curve, v is the design speed and t is the time, then c is given as:

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

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

The value of R for length L_s is given by:

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

Therefore,

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

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

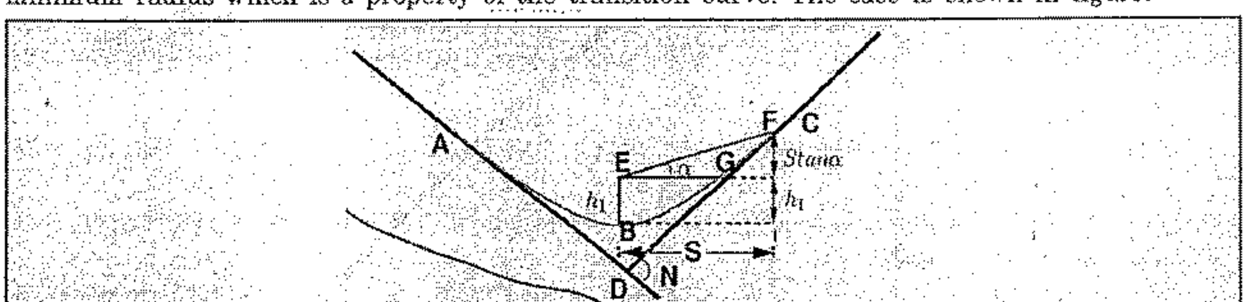
Where L is the total length of valley curve, N is the deviation angle in radians or tangent of the deviation angle or the algebraic difference in grades, and c is the allowable rate of change of centrifugal acceleration which may be taken as 0.6m/sec^3.

Safety criteria

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

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

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



From the geometry of the figure, we have: (assuming small deviation angle, $y = \frac{N}{2L}x^2$)

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

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

where N is the deviation angle in radians, h_1 is the height of headlight beam, α is the head beam inclination in degree and S is the sight distance. The inclination $\alpha \approx 1$ degree and $h_1 \approx 0.75$ as per IRC code.

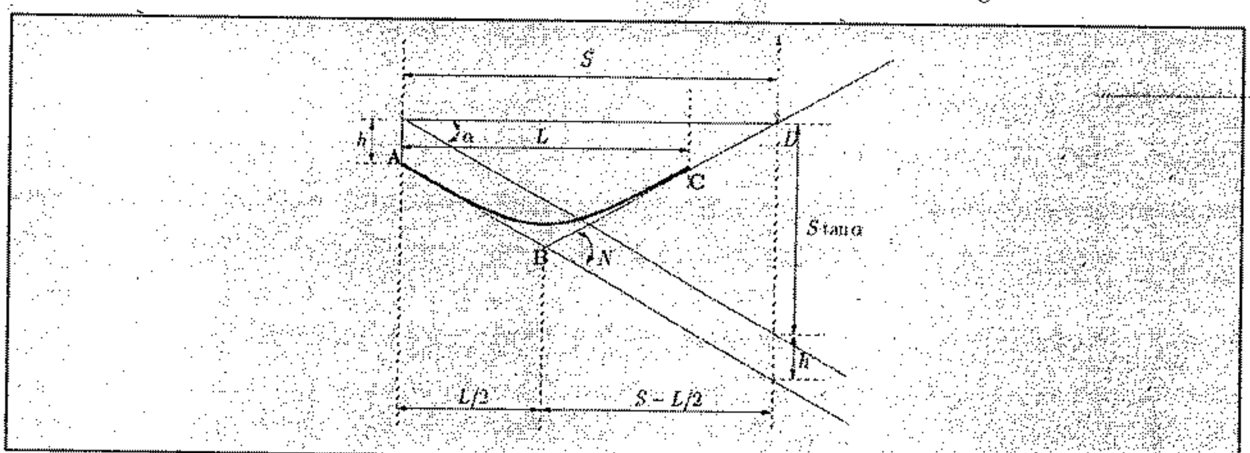
Hence $$L_{(m)} = \frac{NS^2}{1.5 + 0.035S}$$

where $$S = \text{SSD in m}$$

While calculating SSD in this case neglect the effect of slope because minimum sight is at lowest point and beyond that there is gradient.

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

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



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

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

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

For $$\boxed{h_1 = 0.75 \ \& \ \alpha = 1°}$$

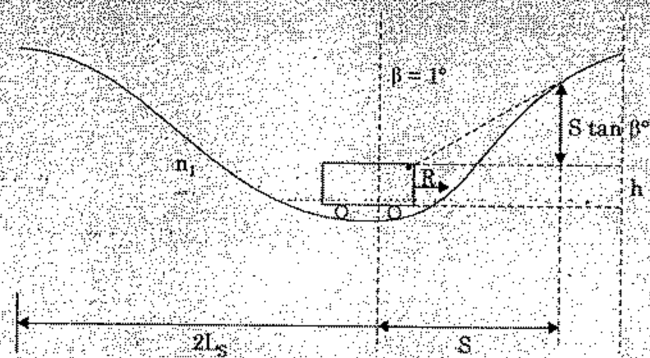
$$L = 2S - \frac{1.5 + 0.035S}{N}$$

While calculating SSD, we should neglect the effect of grade because SSD will be both in up and down grades.

Example 27

A valley curve of a state highway is formed by a descending gradient of 1 in 20 meeting an ascending gradient of 1 in 30. Design the length of a vally curve to fulfil both comfort condition and head light sight distance required for a design speed of 80 km/hr. Assume allowable rate of change of centrifugal acceleration C = 0.60 m/sec³. Suggest the best suitable shape of the valley curve. Consider Reaction time t = 2.5 sec and coeffn of longitudinal friction $f = 0.35$.

Sol. Data given: $n_1 = -\frac{1}{20}$; $n_2 = \left(+\frac{1}{30}\right)$; V = 80 km/hr ; C = 0.60 m/sec³ ; t = 2.5 sec. ; $f = 0.35$



$$\text{Deviation Angle, } N = (n_1 - n_2) = \frac{-1}{20} - \frac{1}{30} = \frac{-3-2}{60} = \frac{-1}{12}$$

Length of valley curve

(a) As per comfort condition,

$$L = 2\left[\frac{NV^3}{C}\right]^{1/2} = 2\left[\frac{1}{12} \times \frac{(0.278 \times 80)^3}{0.60}\right]^{1/2} = 78.17 \text{ m.}$$

(b) Neglecting the ascending and descending gradients at the valley curve

$$\therefore \quad S = 0.278Vt_R + \frac{(0.278 \times V)^2}{2gf}$$

$$= 0.278 \times 80 \times 2.5 + \frac{(0.278 \times 80)^2}{(2 \times 9.81 \times 0.35)} = 127.63 \text{ m}$$

If, L > S.S.D,

$$L = \frac{NS^2}{(2H + 2S\tan\beta)}, \text{ where } \beta = 1° \text{ (Beam Angle)}$$

As per IRC, $2H = 1.5\text{m}.$

$$L = \frac{1}{12} \times \frac{(127.63)^2}{(1.5 + 0.035 \times 127.63S)} = 227.09\text{m} > 127.63\text{m}$$

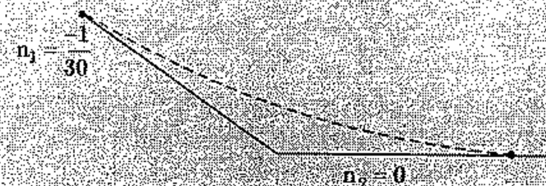
The valley curve length based on headlight sight distance being higher than that based on comfort condition.

Therefore, the length of the valley curve = 227.09m = 228m.

Example 28

Design a valley curve at the junction of a downward gradient of 1 in 30 and a level stretch from head-light consideration. The stopping sight distance is 180m. Treating the curve as a square parabola, set out the curve. (starting point is at the R.L = 10.00m.)

Sol.



Data given, $n_1 = \frac{-1}{30}$; $n_2 = 0$

$\therefore$ $N = n_2 - (-n_1) = \frac{1}{30}$

S.S.D = 180 m.

Assume L > S.S.D

Therefore we know that, $L = \dfrac{NS^2}{(2h + 2S\tan\beta)}$

Put h = 0.75 m.; β = 1°, $L = \dfrac{NS^2}{(2h + 2S\tan\beta°)} = \dfrac{NS^2}{(1.5 + 0.035S)}$

$$= \frac{1 \times (180)^2}{30(1.5 + 0.035 \times 180)} = 138.46\text{m} < \text{S.S.D}$$

Hence our assumption is wrong.

Provide the length of valley curve $= 2S - \dfrac{(1.5 + 0.035S)}{N}$

$$= \left(2 \times 180 - \frac{1.5 + 0.35 \times 180}{\left(\frac{1}{30}\right)}\right) = 126\text{m}.$$

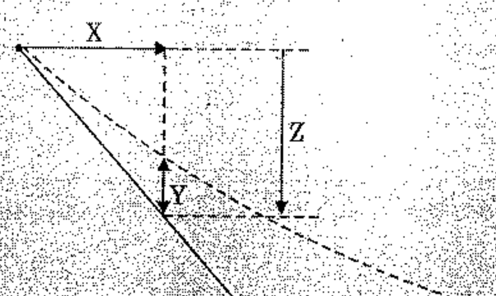
Provide a length of valley curve = 126m.

Shape of valley curve is square parabola.

$$y = \left(\frac{Nx^2}{2L}\right) = \frac{0.033x^2}{(2 \times 126)}$$

The value of y for the various station and R.L of the stations

Station	x(m)	z(m)	y(m)	R.L (m) = 10–z + y
1	0	0	0	10
2	30	1	0.119	9.119
3	60	2	0.476	8.476
4	90	3	1.07	8.07
5	120	4	1.908	7.908
6	126	4.2	2.1	7.9



OBJECTIVE QUESTIONS

1. The maximum super elevation to be provided on a road curve is 1 in 15. If the rate of change of super elevation is specified as 1 in 120 and the road width is 10 m, then the minimum length of the transition curve on either end will be
   (a) 180 m (b) 125 m
   (c) 80 m (d) 30 m

2. A summit curve is formed at the intersection of a 3% up gradient and 5% down gradient. To provide a stopping distance of 128 m, the length of summit curve needed will be
   (a) 271 m (b) 298 m
   (c) 322 m (d) 340 m

3. Consider the following statements:
   Assertion (A): Ideal shape of transition curve should be such that the rate of change of centrifugal acceleration is constant.
   Reason (R): In an ideal transition curve, the length (along the curve) is inversely proportional to the radius.

   Of these statements:
   (a) both A and R are true and R is the correct explanation of A
   (b) both A and R are true but R is not a correct explanation of A
   (c) A is true but R is false
   (d) A is false but R is true

4. Consider the following statements:
   Assertion (A): For mixed traffic conditions, the super elevation should fully counteract the centrifugal force for the full design speed.
   Reason (R): Superelevation needed to maintain the design speed in full may exceed the limiting value of 0.07. Further, as it is not possible to increase the radius, the speed has to be restricted.
   Of these statements:
   (a) both A and R are true and R is the correct explanation of A
   (b) both A and R are true but R is not a correct explanation of A
   (c) A is true but R is false
   (d) A is false but R is true

5. **Assertion (A):** Rotating parts of the surface of the road with the crown as the pivot is not generally preferred for achieving the needful cant at horizontal curves.
   Reason (R): Lowering of the lower edge interferes with the drainage system of the road.

6. Which of the following are the criteria associated with the design of sag vertical curve?
   1. Provision of minimum stopping distance during day time.
   2. Adequate drainage.
   3. Comfortable operation
   4. Pleasant appearance.

Select the correct answer using the codes given below:

(a) 1, 2 and 4 (b) 2 and 3

(c) 2, 3 and 4 (d) 1 and 3

7. Assuming the safe stopping sight distance to be 80 m on a flat highway selection and with a setback distance of 10 m, what would be the radius of the negotiable horizontal curve?

(a) 800 m (b) 160 m

(c) 80 m (d) 70 m

8. Which one of the following is relevant for the determination of superelevation to be provided in horizontal curves of radius R in m in hill roads, given that the design traffic velocity is V kmph?

(a) $\frac{V^2}{127R}$ (b) $\frac{V^2}{17.5R}$

(c) $\frac{V^2}{225R}$ (d) $\frac{(V+8)^2}{127R}$

9. A vehicle was stopped in two seconds by fully jamming the breaks. The skid marks measured 9.8 metres. The average skid resistance coefficient will be

(a) 0.7 (b) 0.5

(c) 0.4 (d) 0.25

10. Which of the criteria given below are used for the design of valley vertical curves on roads?

1. Rider comfort
2. Headlight sight distance
3. Drainage

Select the correct answer using the codes given below:

(a) 1, 2 and 3 (b) 1 and 3

(c) 2 and 3 (d) 1 and 2

11. Consider the following statements:

Assertion (A): It is general practice to provide cant to the road surface with inner edge forming the pivot point.

Reason (R): It does not change centre line levels which have already been fixed at the design stage.

Of these statements:

(a) both A and R are true and R is the correct explanation of A

(b) both A and R are true but R is not a correct explanation of A

(c) A is true but R is false

(d) A is false but R is true

12. For a circular curve of radius 200 m, the coefficient of lateral friction of 0.15 and the design speed is 40 kmph. The equilibrium super elevation (for equal pressure on inner and outer wheel) would be

(a) 21.3 (b) 7

(c) 6.3 (d) 4.6

13. If a descending gradient of 1 in 25 meets an ascending gradient of 1 in 40, then the length of valley curve required for a headlight sight distance of 100 m will be

(a) 30 m (b) 130 m
(c) 310 m (d) 630 m

14. An ascending gradient of 1 in 100 meets a descending gradient of 1 in 50. The length of summit curve required to provide overtaking sight distance of 500 m will be

(a) 938 m (b) 781 m
(c) 470 m (d) 170 m

15. Which one of the following expressions gives intermediate sight distance as per IRC standards?

(a) 2 SSD (b) $\frac{(\text{SSD}+\text{OSD})}{2}$
(c) $\frac{(\text{SSD}-\text{OSD})}{2}$ (d) 2 OSD

16. Brake is applied on a vehicle which then skids a distance of 16 m before coming to stop. If the developed average coefficient of friction between the tyres and the pavement is 0.4, then the speed of the vehicle before skidding have been nearly

(a) 20 kmph (b) 30 kmph
(c) 40 kmph (d) 50 kmph

17. Total reaction time of a driver does not depend upon

(a) perception time (b) brake reaction time
(c) condition of mind of the driver (d) speed of vehicle

18. Which one of the following dictates the minimum required sight distance in valley curves?

(a) Design speed (b) Height of obstacle
(c) height of driver's eye (d) Night time driving condition

19. While driving at a speed of 30 kmph (with available friction 0.4) down the grade, the driver requires a braking distance twice that required for stopping the vehicle when he travels up the same grade. The grade is

(a) 7% (b) 10.6%
(c) 13.3% (d) 33.3%

20. **Assertion (A):** When a sharp horizontal curve is to be introduced on a road which already has the maximum permissible gradient, the gradient should be decreased.

Reason (R): The gradient should be decreased to compensate for the loss of tractive effort due to the introduction of sharp horizontal curve on the road.

21. **Assertion (A):** The centrifugal ratio decreases along the length of the transition curve.

Reason (R): The superelevation increases along the length of the transition curve.

22. Consider the following factors:

1. Reaction time
2. Speed
3. Coefficient of longitudinal friction
4. Gradient

Which of these factors are taken into account for computing braking distance?

(a) 1 and 3 (b) 1, 2 and 4
(c) 2, 3 and 4 (d) 2 and 3

23. Assertion (A): A skillful highway designer 'builds in' speed control at critical locations on horizontal curves rather than increase the superelevation.

Reason (R): A driver slows down on horizontal curve due to feeling of discomfort because of increase in side friction with reduced superelevation.

24. The camber provided on a sloping road is 1 in 48. Which one of the following is the ruling gradient?

(a) 1 in 15 (b) 1 in 20
(c) 1 in 24 (d) 1 in 30

25. Which of the following are the accepted criteria for design of valley curve for highways?

1. Headlight sight distance
2. Passing and non-passing sight distance
3. Aesthetic consideration
4. Motorist comfort
5. Drainage control

Select the correct answer using the codes given below:

(a) 1, 2, 3 and 4 (b) 1, 3, 4 and 5
(c) 2, 3, 4 and 5 (d) 1 and 5

26. For a given road, safe stopping sight distance is 80 m and passing sight distance is 300 m. What is the intermediate sight distance?

(a) 220 m (b) 190 m
(c) 160 m (d) 150 m

27. Consider the following statements:

A transition curve is provided on a circular curve on a highway to provide

1. gradual introduction of centrifugal force
2. minimum stopping sight distance
3. gradual introduction of superelevation
4. comfort and security to passengers

Which of these statements are correct?

(a) 1, 2 and 3 (b) 1, 3 and 4
(c) 2, 3 and 4 (d) 1, 2 and 4

28. What will be the Ruling radius of a horizontal curve on a National Highway for a design vehicle speed of 100 km/h, assuming allowable super elevation to be 7% and lateral friction as 0.13?

(a) 405 m (b) 395 m
(c) 385 m (d) 375 m

29. What happens when the path travelled along the road surface is more than the circumferential movement of the wheels due to rotation?

(a) Slipping (b) Skidding
(c) Turning (d) Revolving

30. Which of the following are requirements for the design of a transition curve for a highway system?

1. Rate of change of grade
2. Rate of change of radial acceleration
3. Rate of change of super elevation
4. Rate of change of curvature

Select the correct answer using the codes given below:

(a) 1, 2 and 3 (b) 2, 1 and 4
(c) 1, 3 and 4 (d) 2, 3 and 4

31. What is the minimum length of a transition curve for a design speed of 80 kmph in a horizontal curve of 240 m radius?

(a) 32 m (b) 42 m
(c) 52 m (d) 72 m

32. The design speed of a highway is 80 km/hr and the radius of circular curve is 150 m in plain topography. Which one of the following is the minimum length of transition curve?

(a) 174 m (b) 85 m
(c) 140 m (d) 120 m

33. A rising gradient of 1 in 50 meets a falling gradient of 1 in 30. Which one of the following is the length of vertical curve if the stopping sight distance is 120 m?

(a) 174 m (b) 158 m
(c) 140 m (d) 120 m

34. If a road surface is adequately superelevated on horizontal curve, which one of the following is the proper distribution of pressure on the vehicle wheels?

(a) Pressure on both outer and inner wheels is equal.
(b) Pressure on inner wheels is less than the outer wheels.
(c) Pressure on inner wheels is more than the outer wheels.
(d) Pressure on front wheel is thrice the pressure on rear wheels.

35. **Assertion (A)**: Road camber helps in surface drainage.
Reason (R): In curved road alignment, super elevation serves the purpose of camber.

36. If super-elevation is not provided on a horizontal curve of a highway, then on which portion of the road, are pot holes likely to develop?

(a) Outer edge of road (b) Inner edge of road
(c) Centre of road (d) Shoulder of road

37. Full amount of super-elevation on a horizontal curve is provided at the

(a) beginning of the transition curve (b) centre of the circular curve
(c) end of the transition curve (d) centre of the transition curve

38. The design speed for a National Highway is 100 kmph. If the maximum permissible superelevation is 0.10. and the coefficient of lateral friction is 0.15, the ruling minimum radius of horizontal curve on the highway should be

(a) 260 m (b) 315 m
(c) 380 m (d) 410 m

39. What is the absolute minimum radius of a curve for safe operation at a speed of 110 kmph?
(a) 110 m (b) 220 m
(c) 440 m (d) 570 m

40. For a road with camber of 3% and the design speed of 80 km/hr, the minimum radius of the curve beyond which no superelevation is needed is
(a) 1680 m (b) 948 m
(c) 406 m (d) 280 m

41. The co-efficient of friction in the longitudinal direction of a highway is estimated as 0.396. The braking distance for a car moving at a speed of 65 km/hr is
(a) 87 m (b) 45 m
(c) 42 m (d) 40 m

42. Which of the following factors affect the required value of camber?
1. The type of pavement surface
2. The amount of rainfall
3. Weight of vehicles
4. Direction of superelevation

Select the correct answer using the codes given below:
(a) 1 and 3 only (b) 3 and 4 only
(c) 1 and 2 only (d) 1, 2, 3 and 4

43. A road is having a horizontal curve of 400 m radius on which a super-elevation of 0.07 is provided. The coefficient of lateral friction mobilized on the curve when a vehicle is travelling at 100 kmph is
(a) 0.07 (b) 0.13
(c) 0.15 (d) 0.4

44. A vehicle moving at 60 kmph on an ascending gradient of a highway has to come to stop position to avoid collision with a stationary object. The ratio of lag to brake distance is 6:5. Considering total reaction time of the driver as 2.5 seconds and the coefficient of longitudinal friction as 0.36, the value of ascending gradient (%) is
(a) 3.3 (b) 4.8
(c) 5.3 (d) 6.8

45. At a horizontal curve portion of a 4 lane undivided carriageway, a transition curve is to be introduced to attain required superelevation. The design speed is 60 kmph and radius of the curve is 245 m. Assume length of wheel base of a longest vehicle as 6 m, superelevation rate as 5% and rate of introduction of this superelevation as 1 in 150. The length of the transition curve (m) required, if the pavement is rotated about inner edge is
(a) 81.4 (b) 85.0
(c) 91.5 (d) 110.2

46. The extra widening required for a two-lane National Highway at a horizontal curve of 300 m radius, considering a wheel base of 8 m and a design speed of 100 kmph is
(a) 0.42 m (b) 0.62 m
(c) 0.82 m (d) 0.92 m

47. The design speed on a road is 60 kmph. Assuming the driver reaction time of 2.5 seconds and coefficient of friction of pavement surface as 0.35, the required stopping distance for two-way traffic on a single lane road is

(a) 82.1 m (b) 102.4 m
(c) 164.2 m (d) 186.4 m

Statement for Linked Answer Questions 48 and 49:

A horizontal circular curve with a centre line radius of 200 m is provided on a 2-lane, 2-way SH section. The width of the 2-lane road is 7.0 m. Design speed for this section is 80 km per hour. The brake reaction time is 2.4 s, and the coefficients of friction in longitudinal and lateral directions are 0.355 and 0.15, respectively.

48. The safe stopping sight distance on the section is

(a) 221 m (b) 195 m
(c) 125 m (d) 65 m

49. The set-back distance from the centre line of the inner lane is

(a) 7.93 m (b) 8.10 m
(c) 9.60 m (d) 9.77 m

50. The value of lateral friction or side friction used in the design of horizontal curve as per Indian Roads Congress guidelines is

(a) 0.40 (b) 0.35
(c) 0.24 (d) 0.15

51. A crest vertical curve joints two gradients of +3% and –2% for a design speed of 80 km/h and the corresponding stopping sight distance of 120 m. The height of driver's eye and the object above the road surface are 1.20 m and 0.15 m respectively. The curve length (which is less than stopping sight distance) to be provided is

(a) 120 m (b) 152 m
(c) 163 m (d) 240 m

52. Which one of the following pairs is correctly matched (Notations have their usual meaning)?

(a) To avoid both skidding and overturning ... $\frac{P}{W} \le \frac{b}{2h} \le f$

(b) Allowable maximum friction coefficient ... 0.15

(c) Allowable coefficient of superelevation ... 0.07

(d) Attainment of superelevation ... $\frac{nl^2}{2R}$

53. The compensated gradient provided at the curve of rad. 60 m with a ruling gradient of 6% is

(a) 5.25% (b) 4.75% (c) 4.5% (d) 3.75%

54. Given that:

Speed of a vehicle = V kmph ; Brake reaction time = t sec ; Efficient of the brakes = η

Then the stopping distance of the vehicle is

(a) $0.28V^2t + \frac{V}{0.01\eta}$ (b) $28Vt + \frac{V^2}{0.1\eta}$

(c) $0.28Vt + \frac{0.01V^2}{\eta}$ (d) $0.28V^2t + 0.01\eta V^2$

55. Consider the following pairs with reference to highway geometric design:

1. Camber for CC pavement ... (1 in 33) to (1 in 40)
2. Roadway formation width ... 12 m for two lane NH in plain terrain
3. Height of the object while calculating stopping sight distance ... 0.15 m
4. Reaction time of driver in the calculation of the overtaking sight distance ... 2.5 s

Which of these pairs are correct?

(a) 2 and 3 (b) 1 and 3 (c) 2 and 4 (d) 3 and 4

56. For a design speed of 80 kmph, if the deviation angle of a valley curve is $\frac{1}{20}$, then the length of a curve for comfort consideration is nearly

(a) 30 m (b) 61 m (c) 101 m (d) 122 m

57. At sharp horizontal curves of highways of radius R (in metres), the percentage reduction in gradient provided to compensate the loss of traction force due to curvature is

(a) 50/R (b) 75/R (c) 100/R (d) 125/R

58. Width and the height of centre of gravity of a vehicle negotiating a horizontal curve, are 'b' and 'h' respectively. μ is the coefficient of friction between the road surface and the wheels. If radius of the curve is low and the speed of the vehicle is high, it would overturn before skidding when

(a) $\frac{b}{2h}$ is more than μ (b) $\frac{b}{h}$ is less than μ

(c) $\frac{b}{h}$ is more than μ (d) $\frac{b}{2h}$ is less than μ

59. Assuming a longitudinal coefficient of friction to be 0.4, the resulting retardation of a vehicle being brought to a stop is, nearly

(a) 0.98 m/s^2 (b) 1.95 m/s^2

(c) 2.90 m/s^2 (d) 3.93 m/s^2

60. If the difference in elevation between the edges of a pavement of width 9.0 m and its crown is 7.5 cm, what is the camber of the pavement?

(a) 1 in 60 (b) 1 in 45 (c) 1 in 30 (d) 1 in 15

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

List-I	List-II
A. Summit curve	1. Superelevation runoff
B. Sag curve	2. Setback distance
C. Horizontal curve	3. Headlight sight distance
D. Transition curve	4. Right of way
	5. Passing sight distance

Codes:

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

62. If N is the algebraic difference of grades, S is the headlight sight distance in metres, then the transmission length of a valley curve (following standard codes) should roughly be equal to

(a) $\frac{NS^2}{6}$ (b) $\frac{NS^2}{9.6}$

(c) $\frac{NS^2}{4}$ (d) $\frac{NS^2}{10}$

63. The design speed of a highway is 80 km per hour. Assuming other data as per IRC recommendations, which one of the following is the approximate adopted lag distance?

(a) 55.5 m (b) 66.7 m

(c) 61.2 m (d) 44.5 m

64. Consider the following steps involved in the design of super elevation in practice as recommended by IRC:

1. Calculation of the allowable speed for maximum 'e' and design value of 'f'
2. Calculation of the super elevation for 75% of the design speed
3. Calculation of the value of 'e' and recheck
4. Calculation of the value of 'f' and recheck

The correct sequence of these steps is

(a) 1-2-3-4 (b) 3-4-1-2

(c) 2-3-4-1 (d) 4-3-2-1

65. Ruling gradient on highways as per IRC in plain terrain is

(a) 1 in 30 (b) 1 in 60

(c) 1 in 100 (d) 1 in 200

66. Consider the following statements:

Assertion (A): The sight distance available on road to a driver at any instant depends on the height of the driver's eye above the road surface and also on the height of the object above the road surface.

Reason (R): The sight distance available on a summit curve is that distance measured along the road surface at which an object of height 0.35 m can be seen by a driver whose eye is at a height of 1.20 m from the road surface.

Of the statements:

(a) both A and R are individually true and R is the correct explanation of A.

(b) both A and R are individually true and R is the correct explanation of A

(c) A is true but R is false

(d) A is false but R is true

67. The absolute minimum radius for a horizontal curve designed for a speed of 100 kmph given the permissible values of super elevation 0.08 and coefficient of friction 0.12 will be

(a) 252 m (b) 295 m

(c) 394 m (d) 364 m

68. What is the superelevation for a horizontal highway curve of radius 500 m and speed 100 kmph in mixed traffic condition?

(a) 8.9% (b) 6.2%

(c) 0 (d) 7%

69. Match **List-I** (Type of pavement) with **List-II** (Camber) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Cement concrete	**1.** 4.0%
B. Water-bound macadam	**2.** 3.0%
C. Thin bituminous	**3.** 2.5%
D. Earth	**4.** 2.0%

Codes:

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

70. What is the value of camber rate that should be provided in case of WBM pavement surface in an area of heavy rainfall?

(a) 1 in 30 (b) 1 in 48
(c) 1 in 60 (d) 1 in 72

71. Consider the following statements:

1. An ascending gradient of 1 in 100 meets an ascending gradient of 1 in 120 to form a valley curve.
2. A falling gradient of 1 in 75 meets a falling gradient of 1 in 50 to form a summit curve.
3. The length of summit curve is determined on the basis of headlight sight distance.

Which of these statements is/are correct?

(a) 1 and 2 (b) 2 and 3
(c) 1 and 3 (d) 2 only

72. Consider the following statements:

Excessive camber is not provided on the roads because

1. transverse tilt causes uncomfort
2. of formation of cross ruts
3. of likely toppling over of highly laden bullock carts
4. of higher costs involved

Which of these statements are correct?

(a) 2, 3 and 4 (b) 1, 3, and 4
(c) 1, 2 and 4 (d) 1, 2 and 3

73. For the total reaction time of 2.5 seconds, coefficient of friction 0.35, design speed 80 km/h, what is the stopping sight distance on a highway?

(a) 124 m (b) 132 m
(c) 76 m (d) 56 m

74. If the width of a carriage way is 5.5 m then what is it called?

(a) Single lane (b) Two lanes
(c) Intermediate lane (d) Multi lane

75. What is the full width of the land acquired before finalizing highway alignment, known as?

(a) Width of formation (b) Right of way

(c) Carriage way (d) Road way

76. At highway stretches where the required overtaking sight distance ***cannot*** be provided, it is advisable to incorporate which one of the following?

(a) At least twice the safe stopping sight distance
(b) Half the required overtaking sight distance
(c) One-third the required overtaking sight distance
(d) Atleast three times the safe stopping sight distance

77. A vehicle with track width of 2.5 m and height 3.8 m is moving on a horizontal curved road way. What is the value of stability factor?

(a) 6.3 (b) 1.3

(c) 0.64 (d) 0.32

78. What is the approximate value of headlight sight distance required on valley vertical curves for highways having design speed 65 kmph, coefficient of longitudinal friction 0.36 and reaction time of driver 2.5 s (rounded off value for design)?

(a) 45 m (b) 60 m

(c) 90 m (d) 180 m

79. Consider the following factors for finding length of summit vertical curve:

1. Sight distance requirements
2. Deviation angle
3. Headlight beam distance
4. Drainage

Which of the above factors are relevant?

(a) 1 and 2 (b) 1 and 3

(c) 2 and 3 (d) 1, 2, 3 and 4

80. How much actual superelevation should be provided for the mixed traffic conditions on a horizontal curve of radius 350 m and speed 80 kmph?

(a) 0.08 (b) 0.07

(c) 0.06 (d) 0.05

81. What is the value of "off tracking" while a vehicle is negotiating a curve of radius 40.0 m with a wheel base of 7.0 m?

(a) 0.75 m (b) 0.69 m

(c) 0.61 m (d) 0.52 m

ANSWERS

1.	(c)	2.	(b)	3.	(a)	4.	(d)	5.	(a)	6.	(c)	7.	(c)
8.	(c)	9.	(b)	10.	(a)	11.	(c)	12.	(c)	13.	(b)	14.	(b)
15.	(a)	16.	(c)	17.	(c)	18.	(d)	19.	(c)	20.	(a)	21.	(d)
22.	(c)	23.	(c)	24.	(d)	25.	(b)	26.	(c)	27.	(b)	28.	(b)
29.	(b)	30.	(d)	31.	(d)	32.	(c)	33.	(a)	34.	(a)	35.	(b)
36.	(a)	37.	(c)	38.	(b)	39.	(c)	40.	(d)	41.	(c)	42.	(c)
43.	(b)	44.	(b)	45.	(d)	46.	(c)	47.	(c)	48.	(c)	49.	(c)
50.	(d)	51.	(b)	52.	(a)	53.	(b)	54.	(c)	55.	(a)	56.	(b)
57.	(b)	58.	(d)	59.	(d)	60.	(a)	61.	(b)	62.	(a)	63.	(a)
64.	(c)	65.	(a)	66.	(c)	67.	(c)	68.	(d)	69.	(b)	70.	(a)
71.	(d)	72.	(d)	73.	(a)	74.	(c)	75.	(b)	76.	(a)	77.	(d)
78.	(c)	79.	(a)	80.	(b)	81.	(c)	82.	(c)	83.	(d)	84.	(c)
85.	(a)												

HINTS AND SOLUTIONS

1. (c)

Sol. Length of transition curve is given by

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

[∵ If Inner Edge is rotated]

$$(L_S)_{min} = \left(\frac{1}{15} \times 120 \times 10\right) = 80$$

2. (b)

Sol. When L > S.S.D.

Length of Summit Curve is given by

$$L = \left(\frac{NS^2}{4.4}\right) = \frac{8 \times (128)^2}{100 \times 4.4}$$

$$= 297.89 \text{ m} \approx 298.0 \text{m}$$

5. (a)

Sol. Lowering of the lower edge interferes with the drainage system of the road advantage is that earth work can be balanced.

Rotation of the inner edge is suitable for area of about high rainfall to avoid the drainage problem.

6. (c)

Sol. There is no restriction to sight distance at valley curves during day time.

The length of the valley transition curve is designed based on the two criteria

(i) Comfort criteria

(ii) Safety criteria

(iii) Headlight Sight distance during night

7. (c)

Sol. When $L_C > S$

$$m = R - (R - d)\cos\left(\frac{\alpha}{2}\right) \quad \text{.....} \quad (i)$$

$$\frac{\alpha}{2} = \frac{360S}{2\pi(R-d)} \quad \text{.....} \quad (ii)$$

∴ By Approximation

$$R = \left(\frac{m}{2} + \frac{S^2}{8m}\right)$$

$$R = \left(\frac{10}{2} + \frac{(80)^2}{8 \times 10}\right) = 5 + 80 = 85 \text{m}$$

9. (b)

Sol. We know that

$$L = \left(\frac{u^2}{2gf}\right) \quad \text{...} \quad (i)$$

$$v = u - at$$

$$\Rightarrow \quad 0 = u - gft \quad [\because a = gf]$$

$$u = gft$$

$$L = \frac{g^2f^2t^2}{2gf}$$

$$\Rightarrow \quad f = \frac{2L}{gt^2} = \frac{2 \times 9.8}{9.8 \times 4} = 0.5$$

12. (c)

Sol. For equal pressure on inner and outer wheel $f = 0$

$$e = \frac{v^2}{gR} = \frac{(0.278 \times 40)^2}{(9.81 \times 200)} = 0.063 \times 100 = 6.3$$

13. (b) $N = \left|-\frac{1}{25} - \frac{1}{40}\right| = 0.065$

$$L = \frac{NS^2}{(1.5 + 0.035S)} = \frac{0.065 \times (100)^2}{(1.5 + 0.035 \times 100)}$$

$$= 130 \text{m}$$

14. (b)

Sol. $N_1 = \frac{1}{100} N_2 = -\frac{1}{50}$

$$N = |N_1 - N_2| = \left|\frac{1}{100} - \left|-\frac{1}{50}\right|\right| = \frac{3}{100} = 0.03$$

$L_C > 0.S.D.$

The length of summit curve

$$l_s = \frac{NS^2}{9.6} = \frac{0.03 \times (500)^2}{9.6} = 781.25 \text{ m}$$

15. (a)

Sol. I.S.D. = 2 S.S.D.

16. (c)

Sol. $L = \frac{v^2}{2gf}$

$$v = \sqrt{2gLf} = \sqrt{(2 \times 9.81 \times 16 \times 0.4)} \times \frac{18}{5}$$

$$= 40.34 \text{ km/hr}$$

17.

Sol. Total reaction time of a driver does not depend upon the condition of mind of the driver.

19. (c)

Sol. $L_1 = 2L_2$

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

$$\Rightarrow \quad f+n = 2f-2n$$

$$\Rightarrow \quad 3n = f$$

$$n = \frac{f}{3} = \frac{0.40}{3} = 0.133 \ = 13.3\%$$

20. (a)

Sol. For R > 45 m, grade compensation $= \frac{75}{R}\%$

The grade compensation in % is $\left(\frac{30+R}{R}\right)$

subject to maximum $\left(\frac{75}{R}\right)$

In other words,

For $R \le 45\text{m}$, Grade Compensation

$$= \left(\frac{30+R}{R}\right)\%$$

21. (d)

Sol. The centrifugal Ratio $\frac{P}{W} = (e+f)$

On transition curve e increases from zero to full value so centrifugal ratio increases

The centrifugal force increases on transition curve gradually.

26. (c) $\text{ISD} = 2\text{ SSD} = 2\times 80 = 160\text{m}$

27. (b)

Sol. A transition curve is provided on a circular curve on a highway to provide

(i) Gradual introduction of centrifugal force

(ii) Gradual introduction of S.E.

(iii) Comfort and security to passengers

28. (b)

Sol. $$R_{ruling} = \frac{v^2}{g(e+f)} = \frac{(0.278\times 100)^2}{9.81(0.07+0.13)} = 393.90 \text{ m}$$

30. (d)

Sol. The requirements for the design of a transition curve (i) ratio of change of radial acceleration

$$c = \frac{v^3}{R\times L}; L = \left(\frac{v^3}{R\times c}\right)$$

(ii) $L = lN(W+W_e)$

(iii) Rate of change of curvature

$$L_S = \left(\frac{2.7v^2}{R}\right) \text{ where V is in km/hr}$$

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

31. (d)

Sol. $L = \left(\frac{v^3}{R\times c}\right)$

$$\text{where } c = \left(\frac{80}{75+v}\right) = \left(\frac{80}{75+80}\right) = 0.516$$

$$L = \frac{(0.278\times 80)^3}{(0.516\times 240)} = 88.82\text{ m}$$

$$L = 2.7\left(\frac{V^2}{R}\right) = 2.7\times\frac{(80)^2}{240} = 72.00$$

Adopt maximum of two

32. (c)

Sol. $$C = \frac{80}{(75+V)} = \frac{80}{(75+80)} = 0.516$$

$$L = \left(\frac{v^3}{R\times C}\right) = \frac{(0.278\times 80)^3}{(150\times 0.516)} = 142.12\text{m}$$

$$L = \frac{2.7v^2}{R} = \frac{2.7\times(80)^2}{150} = 115.2\text{m}$$

So adopt maximum of two *i.e.*, 142.12

33. (a) $L_C > \text{S.S.D.}$

$$N_1 = \frac{1}{50} \quad N_2 = \frac{1}{30}$$

$$N = |N_1 - N_2| = \left(\frac{1}{50}+\frac{1}{30}\right) = 0.053$$

$$L = \frac{NS^2}{4.4} = \frac{0.053\times(120)^2}{4.4} = 174.55\text{m}$$

34. (a) If the road surface is kept horizontal across the alignment, the pressure on the outer wheels will be higher due to the centrifugal force acting outwards and hence the reaction at the outer wheel would be higher.

In order to counter act this centrifugal force and to reduce the tendency of the vehicle to overturn or skid, S.E. is provided.

38. (b)

Sol. $R_{min} = \frac{V^2}{g(e+f)} = \frac{(0.278\times100)^2}{9.81(0.10+0.15)} = 315.12 \text{ m}$

39. (c)

Sol. $R_{min} = \frac{(0.278V)^2}{g(e+f)} = \frac{(0.278\times110)^2}{9.81\times(0.15+0.07)}$

$= 433.29 \text{m}$

41. (c)

Sol. $L = \frac{v^2}{2gf} = \frac{(0.278\times65)^2}{(2\times9.81\times0.396)} = 42.03 \text{ m}$

43. (b)

Sol. $(e+f) = \frac{v^2}{Rg}$

$$f = \frac{(0.278\times100)^2}{(9.81\times400)} - 0.07 = 0.13$$

44. (b)

Sol. $0.278Vt_R \Big/ \frac{(0.278V)^2}{2g(f+s)} = \frac{6}{5}$

$$\Rightarrow t_R \times 2g(f+s) = \frac{6}{5}\times(0.278\times0.60)$$

$$\boxed{S = 4.8\%}$$

45. (d)

Sol. $W_e = \left(\frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}}\right)$

$$= \left(\frac{4\times(6)^2}{2\times245} + \frac{60}{9.5\sqrt{245}}\right) = 0.697 \text{m}$$

for Four Lane

$(W + W_e) = (14 + 0.697) = 14.697 \text{m}$

Length of transition curve = $eN\ (W + W_e)$

$$= \frac{150\times5}{100}\times(14.697)$$

$$= 110.2 \text{m}$$

46. (c)

Sol. Extra widening

$$W_e = \left(\frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}}\right) = \left(\frac{2\times(8)^2}{2\times300} + \frac{100}{9.5\sqrt{300}}\right)$$

$= 0.82 \text{m}$

47. (c)

Sol. For two way single lane = 2 S.S.D.

$$S.S.D = (0.278V_R t) + \frac{(0.278\times v)^2}{2gf}$$

$$= (0.278\times60\times2.5) + \frac{(0.278\times60)^2}{(2\times9.81\times0.35)}$$

$= 82.2 \text{m}$

Required stopping distance

$= 2 \text{ S.S.D}$

$= (2\times82.2) = 164.4 \text{m}$

48. (c)

Sol. S.S.D. $= Vt_R + \frac{(0.278V)^2}{2gf}$

$$= (0.278\times80\times2.4) + \frac{(0.278\times80)^2}{(2\times9.81\times0.355)}$$

$= 124.38 \text{m} = 125 \text{m}$

49. (d)

Sol. $L_C > S.S.D$

$$\frac{\alpha}{360} = \frac{S.S.D}{2\pi(R-d)}$$

$$\frac{\alpha}{2} = \frac{180\times125}{2\pi\times(200-1.75)} = 18.06$$

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

$$= 200 - (200-1.75)\cos(18.06)$$

$= 11.52 \text{ m}$

From the centre line of inner lane

$= (11.52 - 1.75) = 9.77 \text{ m}$

51. (b)

Sol. When $L_C < S.S.D$

Length of vertical curve is given by

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

$$= 2\times120 - \frac{(\sqrt{2\times1.20}+\sqrt{2\times0.15})^2}{(5/100)}$$

$$= \left(240 - \frac{4.4\times100}{5}\right) = 152 \text{ m}$$

53. (b)

Sol. For R > 45 m Grade compensation

$$= \left(\frac{75}{R}\%\right) = \left(\frac{75}{60}\right) = 1.25\%$$

The compensated gradient

$= (6-1.25) = 4.75\%$

56. (b)

Sol. From the comfort condition

$$L_S = 2 \times \sqrt{\left(\frac{NV^3}{C}\right)}$$

$$= 2 \times \sqrt{\frac{0.05 \times (0.278 \times 80)^3}{0.516}}$$

$$= 2 \times 32.64 \quad = 65.2\,m$$

59. (d)

Sol. The resulting retardation

$$a = gf = (9.81 \times 0.4) = 3.924 \text{ m/sec}^2$$

60. (a)

Sol. Camber $= \frac{7.5 \times 10^{-2}}{4.5} = \left(\frac{1}{60}\right)$

63. (a)

Sol. Lag distance = $V_R \times t = (0.278 \times 80 \times 2.5)$

$$= 55.6\,m$$

65. (a)

Sol. It is the maximum gradient that can be provided on road in general condition.

In plain and Rolling → 1 in 30

In steep region → 1 in 16.7

67. (c)

Sol. Absolute minimum Radius

$$R_{min} = \frac{(0.278 \times 100)^2}{9.81 \times (0.08 + 12)} = 394 \text{ m}$$

68. (a)

Sol. For Mixed Traffic Condition

$$l = \left(\frac{v^2}{225R}\right) = \frac{(100)^2}{(225 \times 500)} = 8.89\% > 7\%$$

So adopt 7%

70. (a)

Sol. In case of WBM pavement surface in an area of heavy rainfall camber rate = 3%

73. (a)

Sol. $S.S.D = V_R t + \frac{(0.278v)^2}{2gf}$

$$= (0.278 \times 80 \times 2.5) + \frac{(0.278 \times 80)^2}{(2 \times 9.81 \times 0.35)}$$

$$= 127.63 \text{ m}$$

77. (c)

Sol. The value of stability factor $= \left(\frac{b}{2h}\right) = 0.65m$

80. (b)

Sol. For Mixed Traffic

$$e = \frac{(v)^2}{225R} = \frac{(80)^2}{225 \times 350} = 0.08 > 0.07$$

81. (c)

Sol. Off Tracking

$$= \left(\frac{l^2}{2R}\right) = \frac{(7)^2}{(2 \times 40)} = 0.6125\,m$$

84. (c)

Sol. In Hill Roads, $e = \frac{V^2}{225R}$

CHAPTER

3

Traffic Engineering

INTRODUCTION

- The basic objective of traffic engineering is to achieve free & rapid flow of traffic with least no. of accidents. For this various studies are carried out. These studies are divided into
  - (a) Traffic characteristics
  - (b) Traffic studies and analysis
  - (c) Traffic control regulation
- Based on these studies traffic planning & geometrical design will be done.

TRAFFIC CHARACTERISTICS

- Study of traffic characteristics is the most important, for any improvement of traffic facilities.
- In traffic characteristics, we generally study
  - (a) Road user characteristic
  - (b) Vehicular characteristic
  - (c) Breaking characterstic

Road user characteristics

- It is important to study the characteristics and limitations of road users because the physical, mental and emotional characteristics of human beings affect their ability.
- Factors affecting Road user characteristics are
  - (i) **Physical** : Vision, hearing, strength and General reaction to traffic situations.
  - (ii) **Mental** : Knowledge, skill, intelligence, experience and literacy.
  - (iii) **Psychological** : Attentiveness, fear, anger, superstition, impatience, general attitude towards traffic and regulations and maturity.
  - (iv) **Environmental** : Facilities to the traffic, atmospheric condition and locality.

VEHICULAR CHARACTERISTICS

The study of vehicular characteristics affects the design and traffic performance.

(i) Vehicle dimensions

- Vehicle Dimensions mainly considered are the overall width, height, and length of different vehicles, particularly of the largest ones.
- The width of the vehicle affects the width of the traffic lanes, shoulders and parking facilities.
- Height of the vehicle affects the clearance to be provided under structures such as overbridges underbridges, electric and other service lines.
- Length of the vehicle is an important factor in the design of horizontal alignment as it effects the extra width of pavement and minimum turning radius. Length affects the safe overtaking distance, capacity of a road and parking facilities.

(ii) Weight of Loaded Vehicle

The maximum weight of loaded vehicle affects the design of pavement thickness and gradients. In fact the limiting gradients are governed by both the weight and power of the heavy vehicles.

(iii) Power of vehicle

The power of the heavy vehicles and their loaded weights decides the permissible and limiting values of gradient on roads. The total resistances to traction consisting of inertia, rolling resistance, air resistance and grade resistance.

(iv) Speed of vehicle

Vehicle speed affects, (i) sight distances (ii) superelevation, length of transition curve and limiting radius on horizontal curves (iii) length of transition curves on vertical valley curves and on humps (iv) width of pavement and shoulders on straight and on horizontal curves (v) design gradient (vi) capacity of traffic lane (vii) design and control measures on intersections.

BRAKING CHARACTERISTICS

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

Braking Test

At least two of the following three measurements are needed during the braking tests in order to determine the skid resistance of the pavement:

(i) Braking distance, L metre

(ii) Initial speed, u m/sec.

(iii) Actual duration of brake application, t second.

1. Thus if initial velocity and braking length is known, 'f' can be calculated.

- After the application of brakes, the work done against the frictional force for stopping the vehicle will be equal to the kinetic energy of the vehicle.

$$\frac{1}{2}mu^2 = fWL$$

$$\frac{Wu^2}{2g} = fWL$$

$$\boxed{L = \frac{u^2}{2gf}}$$

where, L = Braking distance, m ; g = Acceleration due to gravity
f = coefficient of friction ; u = initial speed of vehicle.

2. If initial velocity (u) and actual duration of break application is measured,

$$mg.f = ma$$

$$\Rightarrow \quad f = \frac{a}{g} \qquad \text{[a = decleration of vehicle during okidding]}$$

$$a = \frac{u}{t}$$

Hence

$$\boxed{f = \frac{u}{tg}}$$

3. If breaking length and actual duration of break application are measured then

$$0 = u^2 - 2aL$$

$$\Rightarrow \quad a = \frac{u^2}{2L}$$

also

$$a = \frac{f}{g}$$

$$\Rightarrow \quad \frac{u^2}{2L} = \frac{f}{g}$$

$$\Rightarrow \quad \boxed{f = \frac{u^2 g}{2L}}$$

Note : If sometimes the maximum skid resistance is already known than we can find the breaking efficiency also.

$$\eta = \text{breaking efficiency} = \frac{\text{f obtained from breaking test}}{f_{max}\text{ known}}$$

Example 1

Determine the average skid resistance of the pavement surface. During a braking test, a vehicle travelling at a speed of 35 kmph was stopped by applying brakes fully and
(a) Skidmarks were 5.8m in length
(b) Vehicle stopped within 2 sec after application of break.
(c) Vehicles stopped within 1.5 sec and skid marks observed was 7.0 m long.

Sol. (a) Initial speed, $u = \frac{35}{3.6} = 9.722$ m/sec

Braking distace, $L = 5.8\ m = \frac{u^2}{2gf}$

$$5.8 = \frac{9.722^2}{2 \times 9.81 \times f}$$

$$f = 0.831$$

(b) $u = 9.722$

Braking time, $t = 2$ sec

$$v = u + at$$

$$0 = 9.722 + a \times 2$$

$$a = \frac{-9.722}{2} = -4.861\ m/sec^2 \text{ (retardation)}$$

We know that, $F = m\ .\ a$

$$F = \frac{W}{g}.a \quad ; \quad F = W.\frac{a}{g}$$

Average skid resistance, $f = \frac{a}{g} = \frac{4.861}{9.81} = 0.496$

(c) We know that

$v = 0;\ u = 9.722\ ;\ L(s) = 7.0$ m

$$2as = v^2 - u^2$$

$$a = \frac{-u^2}{2S} = \frac{-9.722^2}{2 \times 7} = -6.751 m/sec^2$$

Average skid resistance $= \frac{a}{g} = \frac{6.751}{9.81} = 0.688$

Example 2

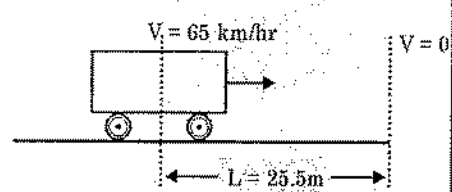
A Vehicle moving at 65 km/hr was stopped by applying brake and the length of skid mark was 25.5 m. If the average skid resistance of the pavement surface is known to be 0.7. Determine the brake effieiency of test vehicle.

Sol.

Given Data : V = 65 km/hr

L = 25.5 m

f = 0.7



We know that Breaking Distance $L = \frac{v^2}{(2gf)}$

$$\therefore \quad f_{obtained} = \left(\frac{v^2}{2gL}\right) = \frac{(0.278 \times v)^2}{2g \times L} = \frac{(0.278 \times 65)^2}{(2 \times 9.81 \times 25.5)} = 0.6526$$

$$\text{Brake Efficiency} = \frac{f_{obtained}}{f_{max}} \times 100 = \left(\frac{0.652}{0.70} \times 100\right) = 93.2\%$$

Example 3

In above Question find out the retardation and time taken to stop the vehicle.

Sol.
$$a = \left(\frac{0-v}{t}\right)$$
$$a = \left(\frac{v}{t}\right) \quad \text{...(i)}$$
$$L = vt - \frac{1}{2}at^2 = vt - \frac{1}{2}\left(\frac{v}{t}\right)\times t^2$$
$$L = \left(\frac{v\times t}{2}\right)$$
$$\Rightarrow \quad \frac{v^2}{2gf} = \frac{v\times t}{2}$$
$$\Rightarrow \quad \left(\frac{v}{t}\right) = gf$$
$$\therefore \quad a = (g \times f)$$

$\therefore$ Retardation = (acceleration due to gravity) × (coefficient of friction) $= (9.81\times 0.652)$
$$= 6.396\,\text{m/sec}^2$$
$$\therefore \quad \text{Time taken, } t = \left(\frac{v}{a}\right) = \frac{(0278\times 65)}{(6.396)}$$
$$t = 2.82\,\text{sec}$$

Example 4

If a vehicle travelling with 85 km/hr speed was stopped in 2.5 sec. Find out (a) Distance travelled after brake (b) Retardation (c) Average skid resistance

Sol. Given Data : V = 85 km/hr = 0.278 × 85 = 23.63 m/sec

t = 2.5 sec

(a) Distance travelled after Brake
$$L = \left(\frac{v\times t}{2}\right) = \left(\frac{23.63\times 2.5}{2}\right)$$
$$L = 29.53\,\text{m}$$

(b) Retardation $a = (g\times f) = \left(\frac{v}{t}\right) = \left(\frac{23.63}{2.5}\right)$
$$a = 9.452\,\text{m/sec}^2$$

(c) Average skid resistance
$$f = \left(\frac{v^2}{2gL}\right) = \frac{(23.63)^2}{(2\times 9.81\times 29.53)} = 0.96$$

Example 5

A vehicle moving at 40 kmph speed was stopped by applying the brake and the length of skid mark was 12.2 m. If the average skid resistance of the pavement is known to be 0.70, determine the brake efficiency of the test vehicle.

Sol.

$$u = \frac{40}{3.6} = 11.11 \text{ m/sec}, L = 12.2 \text{ m}, f = 0.70$$

Average skid resistance developed

$$f = \frac{v^2}{2gL} = \frac{11.11^2}{2 \times 9.8 \times 12.2} = 0.516$$

$$\text{Brake efficiency, \%} = \frac{100f}{f'} = \frac{100 \times 0.516}{0.70} = 73.7\%$$

TRAFFIC STUDIES

- The traffic surveys for collecting traffic data are also called traffic census.
- These studies help in deciding the geometric design feature and traffic control for safe and efficient traffic movements.

The various traffic studies generally carried out are:

(a) Traffic volume study

(b) Speed studies : (i) Spot speed study (ii) Speed and delay study

(c) Origin and destination (O & D) study

(d) Traffic flow characteristics

(e) Traffic capacity study

(f) Parking study

(g) Accident studies

TRAFFIC VOLUME STUDY

- Traffic volume is the number of vehicles crossing a section of road per unit time.
- It is used to measure the quantity of traffic flow (vehicle/day or vehicles/hour).

  It can be determined by ....

$$q = \frac{n \times 3600}{T} \text{vph}$$

where, n = the number of vehicles passing a point in the roadway in T (sec).

q = The equivalent hourly flow

Complete traffic volume study includes : Classified volume study, (i.e. no. of different types of vehicle), Directional study (distribution on different lanes)

- Following are the means of conducting traffic surveys : By toll plaza ticketing, Registration offices, Statistical approach, By interview, By checkpost, Modern Global Positioning Studies.

Uses and objectives of traffic volume studies

(i) It provides a true measure of the relative importance of roads and decides the priority of improvement and expansions.

(ii) It is used in planning of traffic operations, control of existing facilities and designing of new facilities.

(iii) It helps in analysis of traffic patterns and trends.

(iv) It provides useful data for structural and geometric design of pavements and computation of road capacity.

(v) It is used in planning one-way streets and other regulatory measures.

(vi) Turning movement study is used in the design of intersections, in planning signal timings, channelization and other control devices.

(vii) Pedestrian traffic volume study is used for planning side walks, cross walks subways and pedestrian signals.

Traffic Volume Calculation

- The traffic flow varies from time to time as hourly traffic volume varies considerably during day, hourly volume will be much higher than the average hourly volume.
- Daily traffic volumes vary in a week and during seasons.
- Therefore true picture is to be obtained for patterns of hourly, weekly, daily and seasonal variation including class of traffic (buses, truck, cars, Motorcycles and etc.)
- During traffic volume study traffic variations and the direction of each class of traffic is recorded with the information of their turning movements.

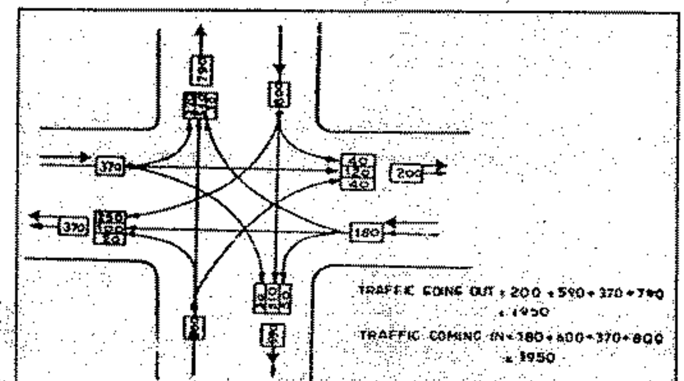


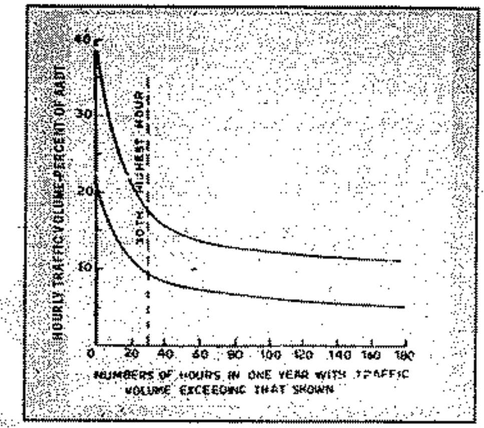
Fig. A Traffic Study data at intersection.

- Traffic volume counts is done with the help of mechanical counters or manually, using pneumatic tube, multipen recorder.
- At first the fluctuations of traffic volume during the hours of day and the daily variations are observed.
- After that with the help of statistics the peak hourly traffic volumes and average daily traffic volumes are calculated.

PRESENTATION OF TRAFFIC VOLUME DATA

- <u>Annual average daily traffic (AADT)</u> → It is the average 24-hr traffice volume at a given location over a full 365-days. Total traffic and classified traffic are calculated. It helps in deciding the relative importance of a route & road development. It includes seasonal variations also.
- <u>Average Daily Traffic (ADT)</u> → It is the average 24-hr volume at a given location for some period of time less than a year. For this minimum of 7-days count is done to include the daily variation like on Saturday & Sunday.
- <u>Trend chart</u> → Showing volume trends over period of years. These data are useful for planning future expansion, design & regulation.

- Variation charts showing hourly, daily and seasonal variations are also prepared. These help in deciding the facilities and regulation needed during peak traffic periods.
- Traffice Flow map along the route→ The thickness of the lines representing the traffic volume to any desired scale.
- Volume flow diagram → at intersections are drawn indicating traffic volume as shown in figure 'A' are prepared, showing the details of crossing and turning traffic. This data is needed for intersection design.
- Thirtieth highest hourly volume → or the design hourly volume is found from the plot between hourly volume and the number of hours in an year that the traffic volume is exceeded. For this all hourly volumes are arranged in decreasing order and order no. is given to each of them. The data at order no. 30 is the 30th highest hourly volume. The 30th highest hourly volume is the hourly volume that will be exceeded only 29 times in a year and all other hourly volumes of the year will be less than this value.



- The highest or peak hourly volume of the year will be too high that it will not be economical to design the facilities according to this volume and the annual average hourly volume (AAHV) found from AADT will not be sufficient during considerable period of an year.
- Thus, It has been observed that 30th highest hourly volume gives a satisfactory results in terms of performance and is also economic in nature. As per Indian condition, 30th highest hourly volume is 8-10% of AADT. For example if AADT is 2000 veh/day, 30th highest hourly volume will be 160-200 veh/hr.
- If 30th highest hourly volume is adopted for desire than there will be congestion only during 29 hours in the year. The thirtieth highest hourly volume is generally taken as the hourly volume for design.

  In order to obtain certain traffic volume data such as AADT, it is necessary to obtain data continuously. However, it is not feasible to collect continuous data on all roads because of cost involved. To make reasonable estimates of annual traffic volume characteristics on an area wise basis, different types of periodic counts with count durations ranging from 15 minutes to continous are conducted.
- **Periodic Volume Counts** - are used to calculate exapnsion factors needed to estimate the annual traffice volume.

$$\text{Hourly expansion factor} = \frac{\text{Total vol. for 24 hr} - \text{period}}{\text{vol. for particular hour}}$$

$$\text{Daily expansion factor} = \frac{\text{Av. total vol. for a weak}}{\text{Av. vol. for particular day}}$$

$$\text{Monthly expansion factor} = \frac{\text{AADT}}{\text{ADT for particular month}}$$

- Thus if 24 hr count at a location is done and hourly volume is calcualted, we can calculate the hourly expansion factor for each hour.

- For example, if total daily traffic is 2000 and hourly vol. for 7:00 am to 8:00 am is 200 and that of 8:00 am – 9:00 am is 400 than hourly expansion factor for 7:00 am to 8:00 am will be $\frac{2000}{200} = 10$ and that for 8:00 am to 9:00 am will be $\frac{2000}{400} = 5$.
- Similarly, if traffic count is done for a week and total traffice volume is 20000, daily volume of monday is 2000 & that of tuesday is 5000, then daily expansion factor for monday will be $\frac{20000}{2000} = 10$ and that for Tuesday = $\frac{20000}{5000} = 4$.

  Similarly, monthly expansion factor the various months can be calculated

Example 6

A Traffic Engineer urgently needs to determine the AADT on a rural primary road the collected the data shown below on a tuesday during the month of May. Determine AADT of the road

Time Period	Vehicles
7:00 – 8:00 am	400
8:00 – 9:00 am	535
9:00 – 10:00 am	650
10:00 – 11:00 am	710
11:00 – 12:00 am	650

The hourly expansion factor for the various times are as under :

Time Period	Daily expansion factor
7:00 – 8:00 am	2.9
8:00 – 9:00 am	22.05
9:00 – 10:00 am	18.80
10:00 – 11:00 am	17.10
11:00 – 12:00 am	18.52

The daily expansion factor for Tuesday is 7.727 and Monthly expansion factor for May is 1.394.

Sol. 24 hr volume for Tuesday $= \dfrac{(400 \times 29) + (535 \times 22.05) + (650 \times 18.80) + (710 \times 17.10) + (650 \times 18.52)}{5} = 11959$

Adjust 24-hr vol. for Tuesday to an av. volume for the weak

Total 7 -day volume = 11959 × 7.727

Av. 24-hr volume $= \dfrac{11959 \times 7.727}{7} = 13201$

Since the data were collected in May.

$$1.394 = \frac{\text{AADT}}{\text{ADT for May}}$$

$\Rightarrow$ AADT = 1.394×13201=18402

Note: The actual design hourly volume may be decided by choosing the hourly traffic volume diagram as shown above. At times the design hourly volumes are not same as 30th hourly volumes.

SPEED STUDIES

Speed of different vehicles vary with respect to time & space. To represent these variation several types of speed can be defined. These are

1. **Spot Speed :** It is the instantaneous speed of a vehicle at a specified location or instant. Spot Speed is needed to design
   i) Horizontal and vertical curve
   ii) Location and size of signs
   iii) Design of signals
   iv) Accident analysis

   Spot speed is measured using Enoscope, Pressure contact tubes and Loop deflector and doppler radar.

2. **Average Speed :** It is the avg. of spot speed of all vehicles passing a given point on the road.

- There are two types of mean speeds or average speed.
  1. Time mean speeds. 2. Space mean speed

Time mean speed (V_t)

- Time mean speed is the arithematic mean of the speed of the vehicles passing a point on a highway during an interval of time.
- It represents tbe speed distribution of vehicles at a point on the roadway.
- Time mean speed is found by

$$V_t = \frac{\sum_{i=1}^{n} V_i}{n}$$

where, V_t = time-mean speed, kmph

V_i = observed instantaneous speed of i^{th} vehicles, kmph

n = number of vehicles observed

Space mean speed (V_s)

- It is the average speed of vehicles over a certain road length at any time.
- It is obtained by dividing the total distance travelled by two or more vehicles on a section of highway by the total time required by these vehicles to travel that distance.
- Space mean speed is the harmonic mean of the speed of the vehicles passing a point at highway during an interval of time.
- It is the speed used for flow density relationship. (i.e., in traffic flow studies).

$$V_s \frac{nL}{\sum_{i=1}^{n} t_i} = \frac{n}{\sum_{i=1}^{n}\left(\frac{1}{u_i}\right)}$$

For example : If three vehicles are moving at a speed of 20 km/hrs., 40 km/hr & 60 km/hr at an instant and traversing a length of D, then time mean speed = $\frac{20+40+60}{3}$ = 40 km/hr. In this problem if one calculates the average time taken by the three vehicles to traverse a distance of D then, average time of travel = $\frac{1}{3}\left[\frac{D}{20}+\frac{D}{40}+\frac{D}{60}\right]=\sum\frac{D/ui}{n}$

$$\Rightarrow \qquad \text{Space mean speed} = \frac{D}{t_{av}}$$

$$\boxed{\text{Space mean speed} = \frac{D}{\sum\frac{D/u_i}{n}} = \frac{n}{\sum\frac{1}{u_i}}}$$

Thus space means speed is the harmonic mean of speed.

where, V_s = Space mean speed

n = number of vehicle

u_i = speed of the i^{th} vehicle

t_i = time taken by i^{th} vehicles to cross the highway.

L = Length of section of highway.

If the speed study is given in the form of frequency distribution table i.e when no. of vehicles in various speed ranges are given, the time means speed and space mean speed is given by

$$\boxed{\text{Time mean speed} = \frac{\Sigma q_i V_i}{\Sigma q_i}}$$

where q_i = No. of vehicles in i^{th} speed range.

V_i = mean velocity of vehicles in the speed range.

$$\boxed{\text{Space mean speed} = \frac{\Sigma q_i}{\Sigma\frac{q_i}{V_i}}}$$

3. Running speed = $\frac{\text{length of travel}}{\text{total time in which the vehicle was running}}$

Running time excludes stop delays. It is used for Raod condition studies.

4. Journey speed = $\frac{\text{length of travel}}{\text{Total journey time}}$

Total journey time includes the stop delays.

Note: Time mean speed is greater than space mean speed. As Airthematic mean is always greater than Harmonic mean.

- Reciprocal of space mean speed gives the average travel time of all the vehicles.
- Travel time is reciprocal of speed (over a length of section) and is measure of how well is a road network operating.

Example 7

Speed of four vehicles on a two lane highway are 20 km/hr, 35 km/hr, 40 km/hr and 45 km/hr. Calculate the time mean speed and space mean speed. Length of section is 300 m.

Sol. We know that, Time mean speed, $V_t = \dfrac{\sum_{i=1}^{i=n} u_i}{n}$

$$V_t = \frac{20+35+40+45}{4} = 35\text{km / hr.}$$

$$\text{Space mean speed, } V_s = \frac{n}{\sum_{i=1}^{n}\left(\frac{1}{u_i}\right)} = \frac{nL}{\sum_{i=1}^{n} t_i}$$

$$V_s = \frac{4}{\left(\frac{1}{20}+\frac{1}{35}+\frac{1}{40}+\frac{1}{45}\right)} = 31.79 \text{ km/hr}$$

Alternatively,

$$V_s = \frac{nL}{\sum_{i=1}^{n} t_i} = \frac{4\times300}{\left(\frac{300}{20}+\frac{300}{35}+\frac{300}{40}+\frac{300}{45}\right)} = 31.79\text{km / hr}$$

Example 8

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

Speed Range	Frequency or (vol. of flow) (q_i)
2 - 5	1
6 - 9'	4
10 - 13	0
14 - 17	7

Sol.

Speed range (m/s)	Av. speed (Vi) i.e. mean of speed range (m/s)	q_i	$V_i q_i$	$\frac{q_i}{V_i}$
2 - 5	3.5	1	3.5	0.286
6 - 9	7.5	4	30	0.534
10 - 13	11.5	0	0	0
14 - 17	15.5	7	108.5	0.451
		$\Sigma q_i = 12 \cong n$	$\Sigma V_i q_i = 142$ $\cong \Sigma u_i$	$\Sigma = 1.271$ $= \Sigma \frac{1}{V_i}$

$$\text{Time mean Speed} = \frac{\Sigma q_i V_i}{\Sigma q_i} = \frac{142}{12} = 11.83 \text{ m/s}$$

$$\text{Space mean speed} = \frac{\Sigma q_i}{\Sigma \frac{q_i}{V_i}} = \frac{12}{1.271} = 9.44 \text{ m/s}$$

Generally there are two types of speed studies carried out.

(i) Spot speed study (ii) Speed and delay study

Spot Speed Study

- **Uses of spot speed study**

(a) In planning traffic control and in traffic regulations.

(b) In geometric design for redesigning existing highways or for deciding design speed for new facilities.

(c) In accident studies

(d) To calculate the traffic capacity

(e) Decide the speed trends

(f) Compare diverse types of drivers and vehicles under specified conditions.

- Factors affecting spot speed are pavement width curve, sight distance, gradient, pavement unevenness intersections and road side developments. Other factors affecting environmental conditions enforcement, traffic conditions, driver, vehicle and motive of travel.

  Spot speed studies cannot be used to find density because measurements are done at one point only.

  note that $\boxed{\text{density = no.of vehicles per km.}}$

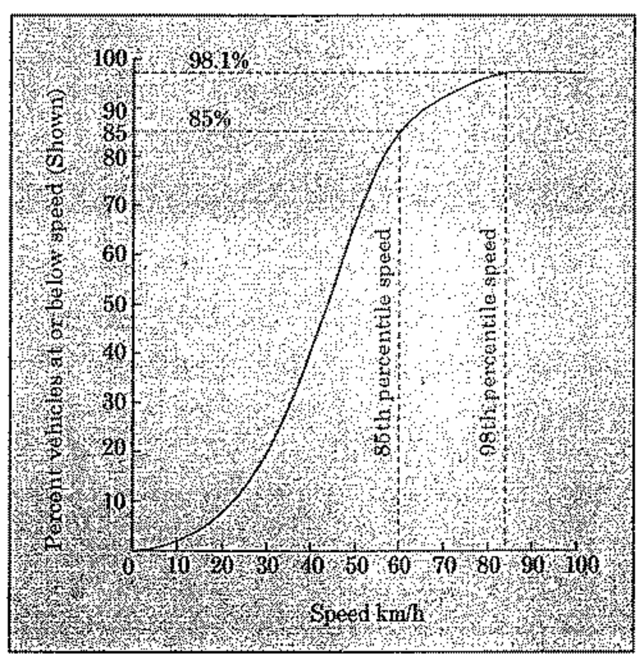
Presentation of spot speed data

(a) Average speed of vehicle i.e. space mean speed and time Mean speed.

- From the spot speed data of the selected samples, frequency distribution tables are prepared for various speed ranges and no. of vehicles in such range. This frequency distribution table prepared gives the general information of the speed distribution pattern. From this space mean & time mean speed are calculated.

(b) Cumulative speed of vehicles

- A graph is plotted between average value of each speed group and cumulative percentage of vehicles travelled at or below different speeds.
- From the above graph, 85th percentile speed is found out which means that only 15 percent of vehicles exceeds this speed during that streach.
- 85th percentile is considered as the safe limit under the zone and all the drivers exceeding this speed (85th percentile speed) are considered faster than safe speed.
- But For the purpose of highway geometric design we consider 98th percentile speed.



- Also, 15th percentile speed is taken as a lower speed limit. It is derived to prohibit the slow moving vehicles to decrease delay and congestion i.e. for segregation of fast moving vehicles with slow moving vehicles.

Example 9

Spot speed studies are carried out at a certain streach of highway and the consolidated data is given below. Prepare a frequency distribution table.

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

Determine (i) the upper and lower values or speed limits for regulation of mixed traffic flow and (ii) the design speed for checking the geometric design elements of the highway.

Sol.

Frequency Distribution of Spot Speed Data

Speed range, kmph	Mid speed, kmph	Frequency, f	Frequency, %	Cumulative Frequency
0-10	5	12	$\frac{12}{850} \times 100 = 1.41$	1.41
10-20	15	18	2.12	3.53
20-30	25	68	8.00	11.53
30-40	35	89	10.47	22
40-50	45	204	24.00	46
50-60	55	255	30.00	76
60-70	65	119	14.00	90
70-80	75	43	5.06	95.06
80-90	85	33	3.88	98.94
90-100	95	9	1.06	100
	Total	850	100.00	

Upper speed limit for regulation = 85th percentile speed $= 55 + \frac{85-76}{90-76} \times 10 = 61.43$ km/hr

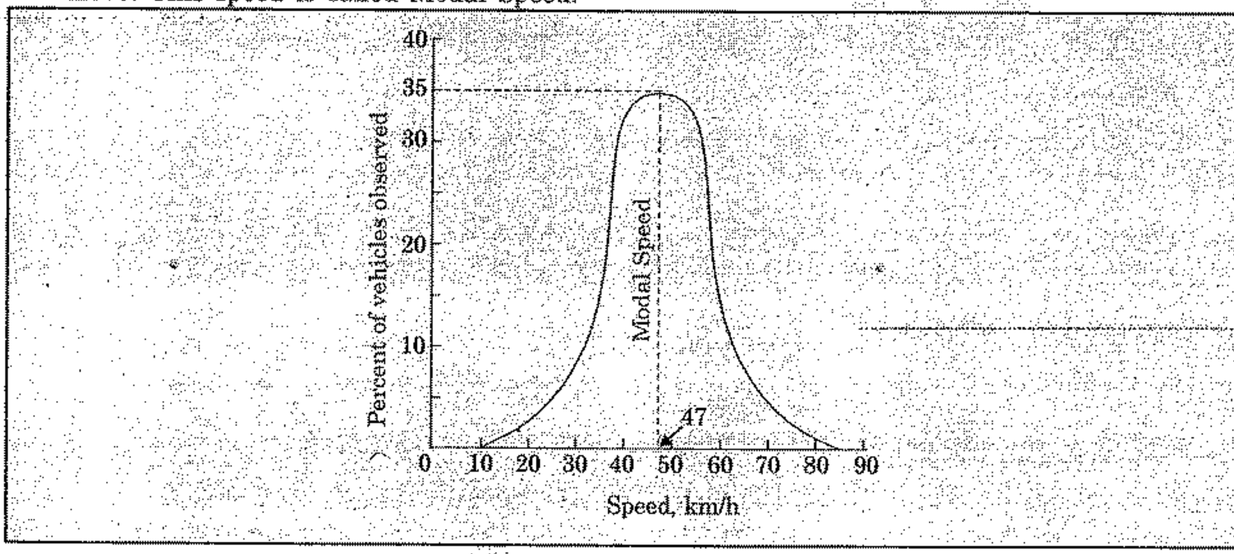
$$\text{Lower limit for regulation} = \text{15th percentile speed} = 25 + \frac{11.53}{22 - 11.53} \times 10 = 28.31 \text{ km/hr}$$

$$\text{Speed for Geometric design} = \text{98th percentile speed} = 75 + \frac{98 - 95.06}{98.94 - 95.06} \times 10$$

$$= 82.57 \text{ km/hr} \approx 83 \text{ km/hr}$$

(c) Modal Average

- A frequency distribution curve of spot speed is plotted with speed of vehicles or avg. values of each speed group of vehicle on the X-axis and the percentage of vehicles in that group on the Y-axis, This graph is called speed distribution curve.
- This curve helps in determining the speed at which the greatest proportion of the vehicles move. This speed is called Modal Speed.



The model speed is 47 km/hr because largest percentage of vehicle are moving at this speed.

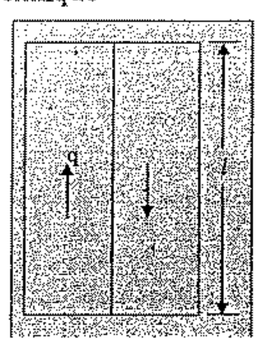
SPEED AND DELAY STUDY

We know that spot speed study canont give the density of traffic. Hence speed & delay studies are carried out over a long distance and hence it is possible to determine density of traffic. This study gives information such as the amount, location and causes of delay in traffic stream. Information of this study can be used in planning and in taking remedia measures to tackle delays at specific locations.

There are various methods of carrying out speed and delay study.
(i) Floating car or riding check method (ii) License plate or vehicle number method
(iii) Interview technique (iv) Elevated observations, (v) Photographic technique

Floating car method

This method is suitable for two lane traffic.



- In floating car method a test vehicle in driven over a given course of travel at approximately the avg. speed of the stream, thus trying to float with the traffic stream. **Four observes are requied in this case.**
- **Observer 1 :** Notes time at various control points such as interaction, bridges, and any other fixed points in each trip. He also notes the amount of delay at each point.

- **2nd Observer :** Notes the time, location and cause of these delays.
- **3rd Observer :** Notes the no. of vehicle overtaking the test vehicle and that overtaken by the test vehicles.
- **4th Observer :** Notes the no. of vehicle travelling in the opposite direction in each trip.

In this case the test is carried out in two directions. Say, when flow & average. journey time of all vehicles is to be calculated in North-South direction, the test is run in N-S as well as S-N direction.

CALCULATION OF FLOW & AVERAGE. JOURNEY TIME IN N-S DIRECTION

If x = No. of vehicles in N-S direction when the test vehicle was moving in South to North direction.

y = No. of vehicles overtaking the test vehicle in N-S direction minus the no. of vehicles overtaken by the test vehicle in N-S direction.

t_w = Journey time of test vehicle in N-S direction

t_a = Journey time of test vehicle in S-N direction

Thus, Average Journey time of traffic in N-S direction is given by

$$\boxed{\bar{t} = \left[t_w - \frac{y(t_a + t_w)}{x + y}\right]}$$

Mean jounry speed of traffic in N-S direction is given by

$$\boxed{V_{mean} = \frac{l}{\bar{t}},\ l = \text{length of travel}}$$

Traffic vol. (q)/flow in N-S direction is given by

$$\boxed{q = \frac{x + y}{t_w + t_a}}$$

$$\text{Running Time} = \left(\bar{t} - \text{Stop delay}\right)$$

$$\text{Running Speed} = \frac{l}{\text{Running time}}$$

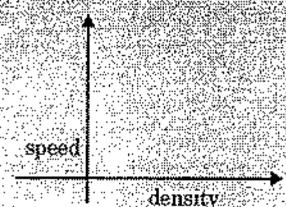
Note : When test vehicle is moving in N-S direction and vehicle from opposite direction is counted (x_2 say) and when test vehicles is moving in South-North direction, vehicle counted in opposite direction (say x_1) then for finding trafic flow etc. in North-South direction we use x as x_1 and for finding traffic flow etc. in **South-North** direction we use x as x_2

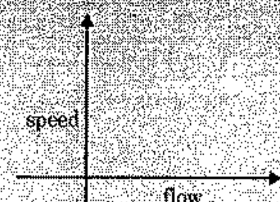
Example 10

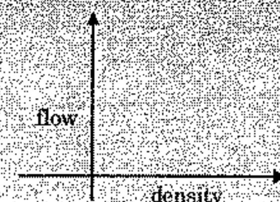
- Length of road strech used = 0.5 Km
- Speed of test vehicle = 20 km/hr.
- No. of vehicles encountered in the stream while the fast vehicle was moving against the stream = x.
- No. of vehicle that had over taken the test vehicle = a.
- No. of vehicle overtaken by test vehicle = b.

Sample No.	x	No. of vehicle over taking test vehicle	No. of vehicle overtaken by test vehicle
1	107	10	74
2	113	25	41
3	30	15	5
4	79	18	9

Find flow, density & average speed of the stream and how the following curves







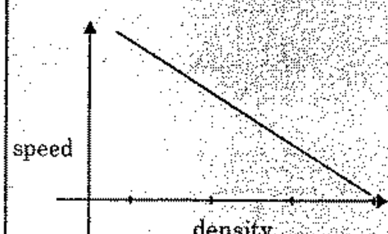
Sol.

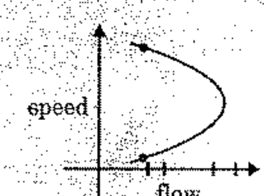
Journey time of test vehicle when moving along stream = tw = $\frac{0.5}{20}$ = 0.025 hr

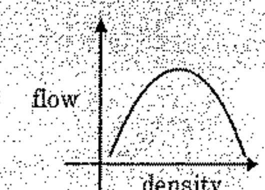
Jounrey time of test vehicle when moving against stream = $t_a = \frac{0.5}{20}$ = 0.025 hr

Sample No.	x	y = (a – b)	ta	tw	$q = \frac{x+y}{ta+tw}$	$\bar{t} = t_w - \frac{y(t_a - t_w)}{x+y} = t_w - \frac{y}{q}$	$\bar{u} = \frac{l}{\bar{t}}$	K
1	107	10 – 74 = –64	0.025	0.025	860	0.0994	5.03	171
2.	113	25 – 41 = –16	0.025	0.025	1940	0.03325	15.04	129
3.	30	15 – 5 = 10	0.025	0.025	800	0.0125	40	20
4.	79	18 – 9 = 9	0.025	0.025	1760	0.01989	25.14	70

$$\text{density (K)} = \text{veh / Km} = \frac{q \text{ veh / hr}}{\bar{u} \text{ km / hr}}$$







ORIGIN AND DESTINATION STUDIES

Objectives of O & D studies

- Origin and destination studies of vehicles determines their numbers, origin and destination in the concerned zone of study.

- It gives information like the actual direction of travel, selection of route and length of trip. Therefore these studies are very helpful in planning new highways and improving new existing services. It is also **used in planning mass rapid transit system.**
- Future and present scientific planning of transportation system and mass transit system should be based on O & D data of passenger trips.

Methods of collection of O & D data

Road side interview method : The vehicles are stopped at predecided interview stations and the answers to prescribed question are is collected on the spot.

- Information collected include the place and time of origin and destination, route, locations of stoppages, the purpose of the trip, type of vehicle and numbers of passengers in each vehicle.
- In this method the data is collected quickly in short duration.
- Main drawback of the method is that the vehicles are stopped for interview.

License plate method

- Entire area under study is marked out and the observers are simultaneously stationed at all points of entry and exit on all the routes leading into and out of the area.
- Each party notes the license plate numbers of the vehicles entering and leaving the marked area and the time.
- After collecting the field data major work remains of the office computations and analysis, by tracking each vehicle number and its time of entering and leaving the marked area.
- This method is quite easy and quick as far as field work is concerned. However involves a lot of office computations in tracing the trips through a net work of stations.
- Hence a large number of teams are required to take simultaneous observations when a large area is to be surveyed.
- This method is quite advantageous when the area under consideration is small, like a intersection or a small business centre.

Return post card method

- Pre-paid business reply post cards with return address are distributed to the road users at some selected points along the route or the cards are mailed to the owners of vehicles.
- Questionnaire to be filled in by the road user is printed on the card, along with a request for co-operation and purpose of the study.
- Distributing stations for the cards may be selected where vehicles have to stop as in case of a toll booth.
- This method is suitable where the traffic is heavy. Only a part of the road users may return the cards promptly after filling in the desired details property and correctly. If conclusions are drawn in such cases, it is likely that these may not give a true picture.

Tag on car method

- A pre-coded card is stuck on the vehicle as it enters the area under study.
- When the car leaves the marked area the other observations are recorded on the tag.
- This method is useful when the traffic is heavy and moves continuously.
- But this method only gives information regarding the points of entry and exit and the time taken to travel the area.

Home interview method

- A random sample of 0.5 to 10 percent of the population it selected and their residences are visited by the trained personal who collect the travel data from each member of the house hold.
- The problem of stopping vehicle and consequent difficulties are avoided.
- Care should be taken in deciding the sample size by keeping in view the desired accuracy .

Work spot interview method

The transportation needs of work trips can be planned by collecting the O & D data at work spots like the offices, factories, educational institutions, etc, by personal interviews.

PRESENTATION OF O AND D DATA

The data are presented in the following forms: (1) O&D Table (2) Desire Lines (3) Pie-chart (4) Contour lines.

(i) Origin and destination tables are prepared showing number of trips between different zones.

(ii) Desire lines are plotted which is a graphical representation prepared in almost all O and D surveys.

- Desire lines are straight lines connecting the origin points with destinations.
- Desire lines show the actual desire of road user and thus helps to find the necessity of a new road link or a diversion or a bypasss.
- The width of such desire lines is drawn proportional to the number of trips in both directions. These desire lines may be compared with the existing flow pattern along the existing routes by superimposing one over the other with the help of tracing sheets.

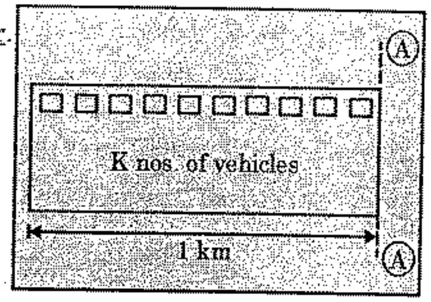
(iii) The relative magnitude of the generated traffic and geometrical relationship of the zones involved may be represented by pie charts, in which circles are drawn, the diameter being proportional to the number of trips.

(iv) Contour lines may be plotted similar to topographic contours. The shape of the contours would indicate the general traffic need of the area.

TRAFFIC CAPACITY STUDIES

- **Traffic volume :** It is the number of vehicles that pass a given point during specified unit of time. It is usually expressed as vehicles/hour or vehicles/day.
- **Traffic density :** It is the number of vehicles occupying a unit length of lane of roadway at

a given instant, usually expressed as vehicles per kilometer.



- Let k no. of vehicles occupy one km of stretch.
- If each vehicle is moving with a velocityof V km/hr., time taken by last vehicle to approach section A – A

$$= \frac{1}{V\,\text{km/hr}}$$

- If q is the traffic volume in vehicle/hr then K no. of vehicles will cross section A–A in time $\frac{K(\text{Veh./Km})}{q(\text{Veh./hr})}$

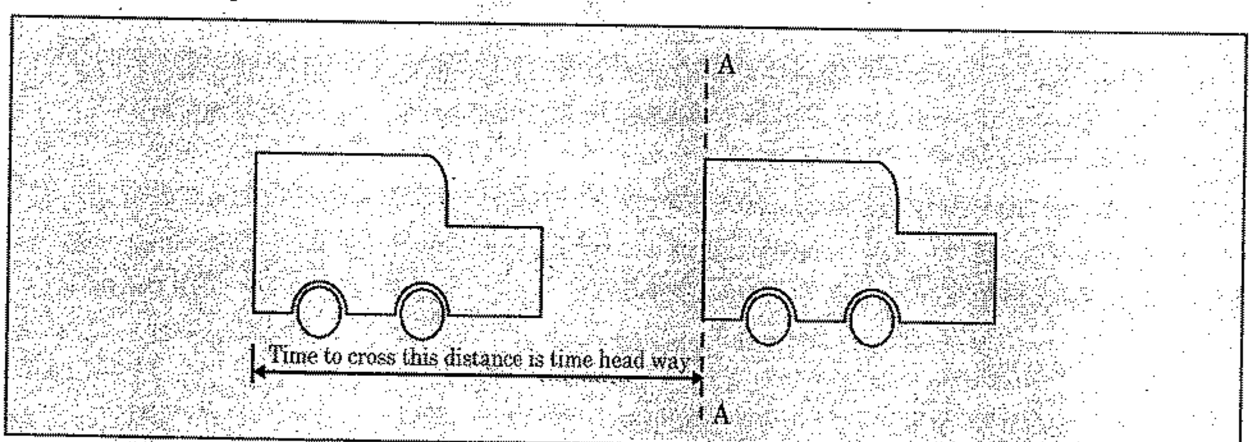
$$\Rightarrow \quad \frac{1}{V} = \frac{K}{q}$$

$$\Rightarrow \quad q = KV$$

i.e. $\boxed{\text{Traffic volume = Traffic density} \times \text{Traffic speed (i.e. space mean speed)}}$

- Highest traffic density will occur when the vehicles are practically standstill on a given route but in this case traffic volume will tend to zero as traffic speed will tend to zero.
- From the traffic flow volume, density & speed, following parameters are defined.

(a) **Time head way (h_i)** → It is the time interval between the passes of rear bumper of successive vehicle at a point.



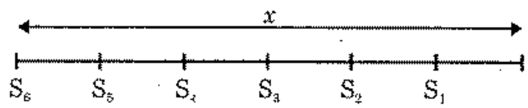
If n_t vehicle cross A – A in time t then

$$\text{Traffic Volume} = q = \frac{n_t(\text{Veh.})}{t(\text{hr})} = \frac{n_t}{\Sigma h_i} = \frac{1}{\frac{\Sigma h_i}{n_t}} = \frac{1}{h_{av}\left(\frac{\text{hr}}{\text{veh}}\right)}$$

$$\Rightarrow \quad \boxed{\text{Traffic Volume (Veh./hr)} = \frac{1}{\text{Av. time headway (hr/Veh)}}}$$

(b) **Space headway : (Si)**

Distance between the rear bumper of succesive vehicles is called space head way. In a particular lane, length of observation is the summation of space head ways.



$\Rightarrow x = \Sigma S_i$

$\Rightarrow$ density per lane $= \frac{n_x}{x}$, [n_x = no. of vehicles in length x in a lane.]

$$\Rightarrow \text{density per lane} = \frac{n_x}{\Sigma S_i} = \frac{1}{\frac{\Sigma S_i}{n_x}}$$

$$= \frac{1}{\text{Av. space headway}}$$

$$\boxed{\text{Density per lane} = \frac{1}{\text{Av. space headway}}}$$

TRAFFIC CAPACITY

It is the ability of a roadway to accommodate traffic volume. Usually expressed as the maximum number of vehicle in a lane or a road that can pass a given point in unit time, usually an hour, *i.e.*, It is expressed as vehicles per hour per lane.

Note: *Traffic capacity and traffic volume are measures of traffic flow and have same units. But traffic capacity represents the capability or maximum rate of flow on a road way provided with certain level of service characteristics where as traffic volume indicates the actual rate of flow and responds to variations in traffic demand.*

- Traffic capacity of roadway depends on a number of prevailing roadways and traffic conditions. Traffic Capacity is Categorised as:

1. **Basic capacity :** It is the maximum number of passenger cars that can pass a given point on a lane or roadway during one hour under the most nearly ideal roadway and traffic conditions which can possibly be attained.

- Basic capacity is the theoretical capacity.

Note: *Two roads having the same physical features will have the same basic capacity irrespective of traffic conditions, as they are assumed to be ideal.*

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2. **Possible capacity :** It is the maximum number of vehicles that can pass a given point on a lane or roadway during one hour under prevailing roadway and traffic conditions.

- Generally possible capacity of roadway is much lower than the basic capacity as the prevailing roadway and traffic conditions are rarely ideal.
- If prevailing condition is bad due to the traffic congestion, the traffic may become standstill and possible capacity of road may approach zero.
- When prevailing conditions are close to ideal conditions, the possible capacity will approach to the basic capacity.
- The value of possible capacity varies between zero to basic capacity.

Note: For design purpose we neither use basic capacity or possible capacity as they represents the two extreme cases of roadway and traffic condition. Hence we have another type of capacity called Practical capacity.

3. **Practical capacity** : It is the maximum number of vehicle that can pass a given point on a roadway during one hour, without traffic density being so great, as to cause unreasonable delay, hazard or restriction to the driver's freedom to manoeuvre under the prevailing roadway and traffic conditions.

Note: Sometimes practical capacity is also called as design capacity.

Calculation of theoretical maximum capacity :

Max Theortical Capacity from Space headway

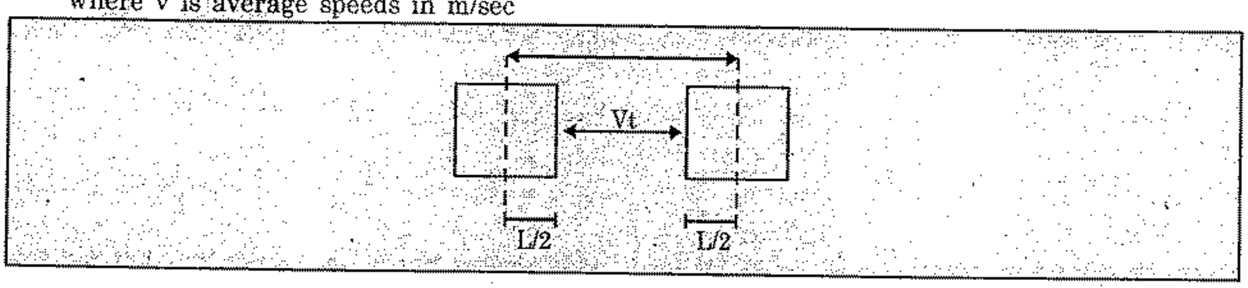
We know that Traffic Volume per lane = Density per lane × Space Mean Speed

$$\Rightarrow \quad \text{Traffic Volume per Lane (q)} = \frac{1}{\text{Space headway}} \times \text{Space mean Speed}$$

- Thus when, space headway is such that it has a minimum value ensuring ideal road way & traffic condition, the traffic volume will be max. This max value of volume is called maximum Theortical Capacity.
- Ideal space headway between end to end or centre to centre of vehicles is equal to the average length of vehicle plus the clear spacing between the vehicles in the stream.
- Minimum clear spacing between vehicles are allowed for safe stopping of the rear vehicle in case the vehicle ahead suddenly stops.
- It is observed that drivers follow the vehicle ahead at a closer gap at lower speed where as spacing increases when the speed is high.
- The space gap allowed by the following vehicle in a traffic stream is some time assumed to be equal to the distance travelled during the reaction time of the driver, under assumption that the braking distances of the leading and the following vehicles are approximately equal. If the reaction time is t sec., the minimum space gap S_g is given by:

$$S_g = vt$$

where v is average speeds in m/sec



- Minimum space headway S in a traffic stream is hence equal to the minimum space gap plus average length of vehicle L in the stream.

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

- In a stream flow, the average reaction time is found to be low; this value is often assumed to be 0.70 to 0.75 sec. Generally the value of reaction time has been assumed as 0.7 sec. in the empirical relation for spacing, i.e.,

$$S = (0.7\ v + L),\ L = \text{Length of Vehicle}$$

Hence a suitable value of S may be adopted to estimate the theoretical capacity of a traffic lane with homogeneous traffic flow.

- Theoretical maximum or basic capacity of a single lane is given as

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

where, C = capacity of a single lane, vehicle per hour

V = speed, kmph

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

Hence, theoretical maximum capacity depends on the speed (v) and spacing (S).

Max Theortical Capacity from Time Headway :

It is observed that with increase in speed of the traffic stream, the time headway decreases and after reaching a minimum value at an optimum speed, starts increasing. Therefore maximum theoretical capacity of a traffic lane may be obtained if the minimum headway H_t is known.

We know that

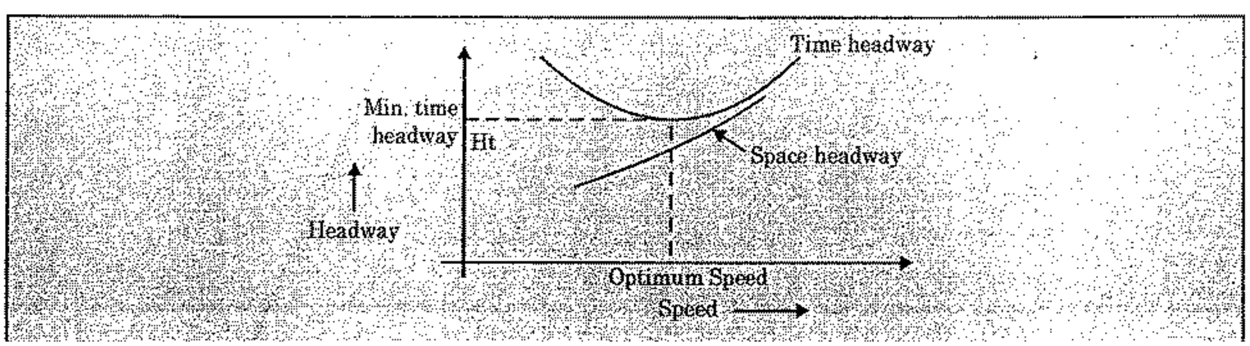
$$\text{Volume} = \frac{1}{\text{Time headway}}$$

$$\Rightarrow \quad \text{Max theortical capacity} = \frac{1}{\text{min time head way (sec/ veh)}}$$

$$\text{Hence Max. Theoritical Capacity} = \frac{3600}{\text{min time headway in sec.}}$$

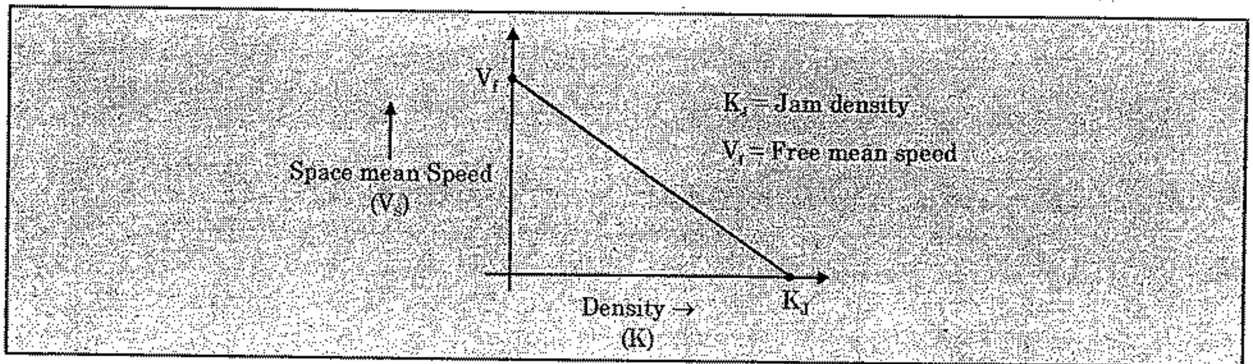
$$q_{max} = \frac{3600}{Ht};\ Ht = \text{min. time headway in second}$$

$$\boxed{C = \frac{3600}{H_t}}$$



Relation between speed density & volume

As per Greenshield, the relationship between the speed and density should be assumed linear.



Note that at Jam density (i.e. max. density) space mean speed will be zero because vehicles will be in stand still conditions. When density is zero i.e. when there is no vehicle on road, the speed will be maximum called free mean speed.

Relation between speed & density is thus given by

$$\boxed{V_s = V_f - \frac{V_f}{K_J}K} \quad \text{.... (i)}$$

As we know that traffic volume (q) = Space mean speed (V_s) × density (K)

Hence $$q = K\left[V_f - \frac{V_f}{K_J}K\right]$$

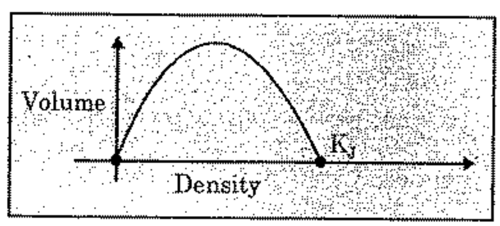
$$\Rightarrow \quad \boxed{q = V_f(K) - \frac{V_f}{K_J}(K^2)} \quad \text{.... (ii)}$$

Thus relation between volume & density will be parabolic.

Similarly,

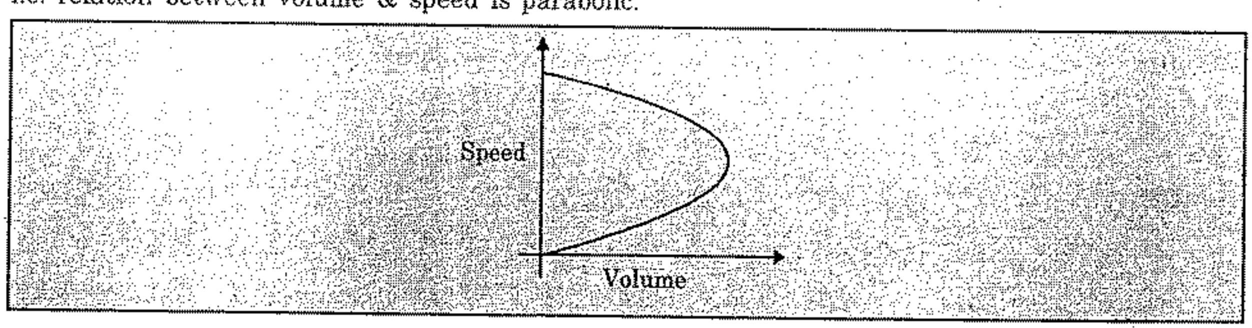
$$q = V_S K$$

$$= V_S\left[(V_f - V_S)\frac{K_J}{V_f}\right], \text{ from (i)}$$



$$\Rightarrow \quad \boxed{q = (K_J)V_S - \left(\frac{K_J}{V_f}\right)V_S^2} \quad \text{.... (iii)}$$

i.e. relation between volume & speed is parabolic.



VARIOUS CONDITIONS

1. **To find density at maximum flow** (i.e. volume)

$$\text{for max volume, } \frac{dq}{dK} = 0$$

$$\Rightarrow \quad \frac{d\left[V_f K - \frac{V_f}{K_J} K^2\right]}{dK} = 0$$

$$\Rightarrow \quad V_f - \frac{2V_f K}{KJ} = 0$$

$$\Rightarrow \quad K = \frac{K_J}{2} \quad \text{......: (iv)}$$

Thus max. flow (i.e. volume) will occur at density equal to half of dam density.

2. **To find max.flow**

$$\text{max flow will occur at } K = \frac{K_J}{2}$$

$$\Rightarrow \quad q = V_f K - \frac{V_f}{K_J} K^2$$

$$q_{max} = V_f\left(\frac{K_J}{2}\right) - \frac{V_f}{K_J}\left(\frac{K_J}{2}\right)^2$$

$$= \frac{V_f K_J}{2} - \frac{V_f K_J}{4}$$

$$\Rightarrow \quad \boxed{q_{max} = \frac{V_f K_J}{4}} \quad \text{(v)}$$

3. To find speed at maximum flow (i.e. volume)

$$\text{for max. flow } \frac{d_q}{dV_S} = 0$$

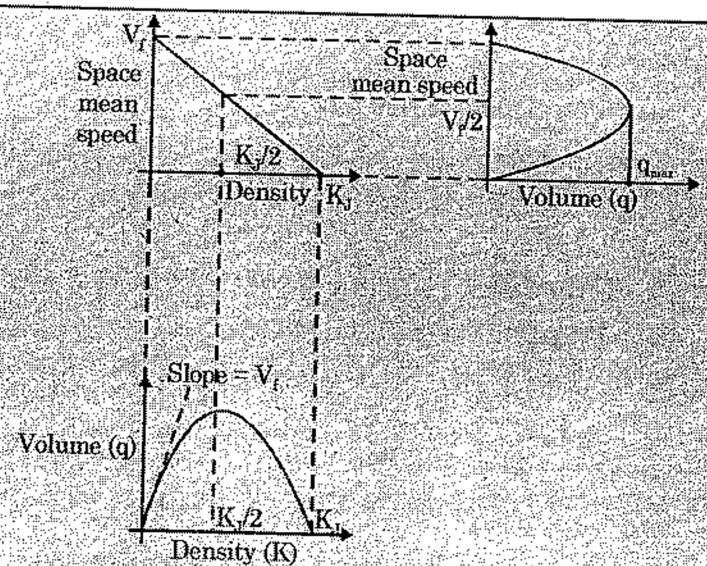
$$\Rightarrow \quad \frac{d\left[K_J V_S - \frac{K_J V_S^2}{V_f}\right]}{dV_S} = 0$$

$$\Rightarrow \quad K_J - \frac{2K_J V_S}{V_f} = 0$$

$$\Rightarrow \quad V_S = \frac{V_f}{2} \quad \text{.......(vi)}$$

Thus for max volume, speed should be half of free mean speed.

Note : The max flow is called capacity flow :



Travel Time

- Travel time is defined as the time taken to complete a journey.
- Travel time *per unit length of road* is inversely proportional to the speed. If T is travel time and V is speed (kmph), then

$$\boxed{T\ (\text{sec/km}) = \frac{3600}{V}}$$

- It is observed from the above graph (Travel time vrs speed) that at higher speed, the rate of saving in travel time decreases.

Example 11

The speed relationship for a particular road was found to be u = (42.76 – 0.22 k) where 'u' is the speed in km/hr and k is the density of vehicle per km. Find jam density, maximum capacity and density at maximum capacity. Sketch the relationship between density and flow and indicate important traffic flow parameters on it.

Sol. We know that, Traffic Volume = (traffic density × speed) $= \left(\frac{\text{Veh}}{\text{km}}\right) \times \left(\frac{\text{Km}}{\text{hr}}\right)$

$$\therefore \quad q = k\,u = k\,(42.76 - 0.22\,k) = (42.76\,k - 0.22k^2)$$

If $\quad q = 0 = (42.76\,k - 0.22\,k^2)$

$$\Rightarrow \quad k(42.76 - 0.22k) = 0$$

$$\therefore \Rightarrow \quad k = 0, \text{ or } k = \left(\frac{42.76}{0.22}\right) = 194.36\,\text{Veh/km} = \text{Jam density}$$

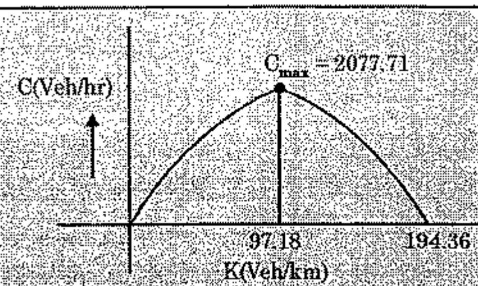
For maximum value of q, $\frac{dq}{dk} = 0$

$$\Rightarrow \quad \frac{dq}{dk} = (42.76 - 0.44k) = 0$$

$$\Rightarrow \quad 0 = (42.76 - 0.44k)$$

$$k = \left(\frac{42.76}{0.44}\right) = 97.18 \text{ Veh/km} = \text{density at maximum capacity}$$

$$\therefore \quad q_{max} = \left(42.76 \times 97.18 - 0.22 \times (97.18)^2\right) = \text{maximum capacity} = 2077.71 \text{ veh/hr}$$



Example 12

In the above problem find out the minimum time headway and minimum space headway.

$$q_{max} = \frac{1}{\text{min. time headway}}$$

$$\Rightarrow \text{Min. time headway} = \frac{1}{q_{max}}$$

$$= \frac{1}{2077.71} \text{ h/veh}$$

$$= \frac{3600}{2077.71} \text{ sec/veh}$$

$$= 1.73 \text{ sec}$$

In this case we have to calculate space headway corresponding to the capacity flow.

$$\frac{1}{\text{Space headway}} = \frac{K_J}{2}$$

$$\text{Space headway at capacity flow} = \frac{2}{k_J} \text{ veh./km}$$

$$= \frac{2000}{194.36} = 10.29 \text{ m}$$

Example 13

The free mean speed on a roadway is found to be 80 km/hr. Under stopped condition the average spacing between vehicles is 6.9 m. Determine the capacity flow.

Sol. Given Data : Free Mean Speed $V_{S_1} = 80$ km/hr

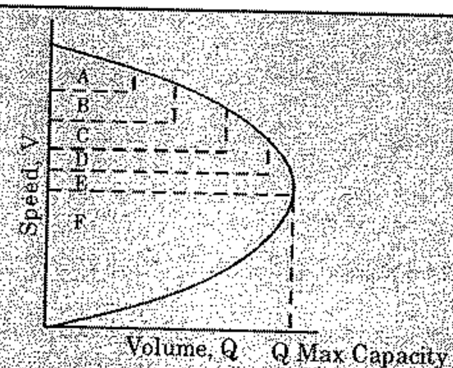
Average Spacing between Vehicle = 6.9 m

$$\text{Jam Density} \quad K_j = \frac{1000}{6.9} = 145 \text{ Vehicle/km}$$

$$\text{Maximum Flow} = \left(\frac{V_{sf} \times K_j}{4}\right) = \left(\frac{80 \times 145}{4}\right) = 2900 \text{ Vehicle/hr}$$

CAPACITY AND LEVEL OF SERVICE

- Capacity standards are fixed normally in relation to the level of service (LOS).
- Capacity is the quantative measure whereas level of service is the qualitative measure of flow.
- Capacity of road depends on geometrical design facility, environmental condition etc.
- Level of service is defined as a qualitative measure describing operational conditions with in a traffic stream and their perception by drivers / passengers.
- Six levels of service are recognized commonly, designated from A to F, with Level of Service A representing the best operating condition (i.e. free flow) and Level of Service F the worst (i.e. forced or breakdown flow).



Speed volume curve showing level of service.

Level of Service A

- Represents a condition of free flow with average travel speeds usually about 90 percent of the free-flow speed for the arterial class.
- Individual users are virtually unaffected by the presence of others in the traffic stream.
- Freedom to select desired speeds and to manoeuvre within the traffic stream is high.
- The general level of comfort and convenience provided to the road users is excellent.

Level of Service B

- Represents a zone of stable flow, with the drivers still having reasonable freedom to select their desired speed and manoeuvre within the traffic stream
- Average travel speeds are usually about 70 percent of the free flow speed for the arterial class.
- Level of comfort and convenience provided is somewhat less than Level of Service A because the presence of other vehicles in the traffic stream begins to affect individual behaviour.

Level of Service C

- This also a zone of stable flow, but marks the beginning of the range of flow in which the operation of individual users becomes significantly affected by interactions with others in the traffic stream.
- The selection of speed is now affected by the presence of others, and manoguvring within the traffic stream requires substantial vigilance on the part of the user.
- The general level of comfort and convenience declines noticeably at this level.
- Average travel speeds are about 50 per cent of the average free flow speed.

Level of Service D

- Represents the limit of stable flow, with conditions approaching close to unstable flow.
- Due to high density, the drivers are severely restricted in their freedom to select desired speed and manoeuver within the traffic stream.
- The general level of comfort and convenience is poor.
- Small increases in traffic flow will usually cause operational problems at this level.
- Average travel speeds are about 40 percent of free flow speed.

Level of Service E

- Represents operating conditions when traffic volumes are at or close to the capacity level.
- The speed are reduced to a low, but relatively uniform values, average value being one-third the free flow speed.
- Freedom to manoeuvre within the traffic stream is extremely difficult, and is generally accomplished by forcing a vehicle to give way to accommodate such maneuvers.
- Comfort and convenience are extremely poor and driver frustration is generally high.
- Operations at this level are usually unstable because small increases in flow or minor disturbances within the traffic stream will cause breakdowns.

Level of Service F :

- Represents zone of forced or breakdown flow.
- This condition occurs when the amount of traffic approaching a point exceeds the amount which can pass it.
- Queues from behind such locations
- Operations within the queue are characterized by stop-and-go waves which are extremely unstable
- Vehicles may progress at a reasonable speed for several hundred meters and may then be required to stop in a cyclic fashion.
- Due to high volumes, breakdown occurs and long queues and delays result
- The average travel speeds are between 25 percent and 33 percent of free flow speed.

Example 14

Estimate theoretical capacity of a highway with one-way traffic flow at 55 km/hr speed. Consider average length of vehicle 6.2 m. Also claculate maximum theoretical capacity if time head-way is 3 sec.

Sol. Given Data : V = 55 km/hr ; L = 6.2 m

Therefore, Minimum spacing between the vehicle

$$s = (0.2V + L) = (0.2 \times 55 + 6.2) = 17.2\text{m}$$

Therefore, Theoretical capacity $= \dfrac{1000V}{S} = \dfrac{1000 \times 55}{17.2} = \dfrac{1000 \times 55}{17.2} = 3197\,\text{Vehicle/hr}$

$\therefore$ Maximum Theoretical capacity $= \left(\dfrac{3600}{H_t}\right) = \left(\dfrac{3600}{3}\right) = 1200\,\text{Veh/hr}$

Passenger Car Unit (P.C.U.)

- It is difficult to estimate the traffic volume and capacity of roadway facilities under mixed traffic flow, unless the different vehicle classes are converted to one common standard vehicle unit.
- A common practice to consider the passenger car as the standard vehicle unit to convert the other vehicle classes and this unit is called Passenger Car Unit or PCU.
- Hence, in mixed traffic flow, the traffic volume and capacity are generally expressed as PCU per hour or PCU/lane/hour and the traffic density as PCU per kilometre length of lane.
- If one vehicle of a particular class in the traffic stream produces the same effect as that due to the addition of one passenger car, then that vehicle class is considered equivalent to the passenger car with a PCU value equal to 1.0.
- PCU value of a vehicle class may be considered as the ratio of the capacity of a roadway when there are passenger cars only to the capacity of the same roadway when there are vehicles of that class only.

Factors affecting PCU values

(a) speed of the vehicle under the prevailing roadway and traffic conditions within the desired speed range.

(b) length and width of the vehicle.

(c) transverse gap and longitudinal gap allowed between the vehicles of the same class in the speed range under consideration, during compact stream flow.

Equivalency factors suggested by the IRC for Rural Road in Sections of Plain Terrain

S.No	Vehicle class	Equivalency Factors
1.	Passenger car, tempo, auto-rickshaw, agricultural tractor	1.0
2.	Bus, truck, agricultural tractor-tailer unit	3.0
3.	Motor cycle, scooter and pedal cycle	0.5
4.	Cycle rickshaw	1.5
5.	Horse drawn vehicles	4.0
6.	Small bullock cart and hand cart	6.0
7.	Large bullock cart	8.0

Practical Capacity Values

The practical capacity values suggested by the IRC for the purpose of design of different types of roads in rural areas are as under.

Capacity of different types of roads in rural areas

Types of road	Capacity PCU per day (both directions)
Single lane with 3.75 m wide carriageway and normal earthen shoulders	1000
Single lane roads with 3.75 m wide carriageway and 1.0 m wide hard shoulders	2500
Road with intermediate lanes of width 5.5 m and normal earthen shoulders	5000
Two lane roads with 7.0 m wide carriageway and earthen shoulders	10,000
Four lanes divided highway (depending on traffic, access control, etc.)	20,000 to 30,000

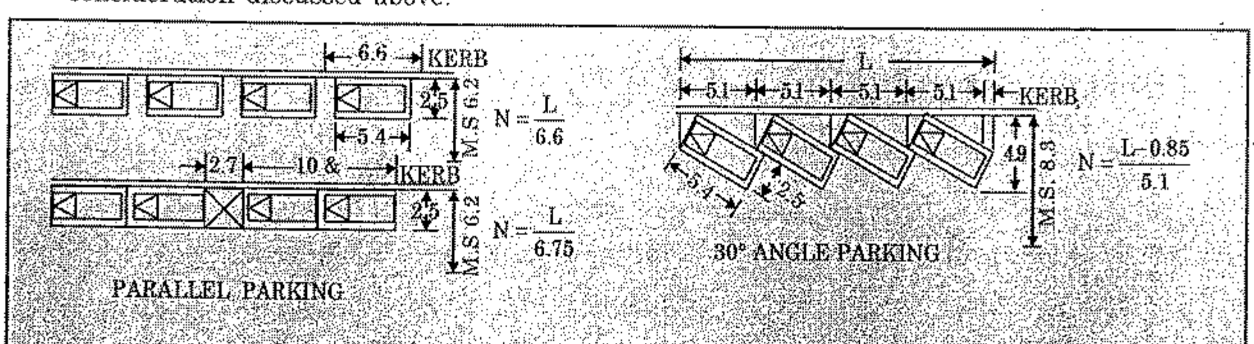
TYPES OF PARKING FACILITIES

There are two basic types of parking facilities.

- On-street or kerb facilities
- Off-street parking facilities

On-Street of Kerb Parking

- In this type of parking vehicles are parked on the kerb which may be designed for parking.
- Angle parking or parallel parking may be allowed in the kerb parking. Angle parking may be at angles 30, 60, or 90 degrees.
- Angle parking accommodates more vehicles per unit length of kerb and maximum vehicles that can be parked is with an angle of 90 degree. Angle parking is more convenient for the motorists than the parallel parking, but it produces much more obstruction to the through traffic resulting in more accidents than the parallel parking.
- Out of various angles used in angle parking, 45 degree angle is considered the best from all consideration discussed above.



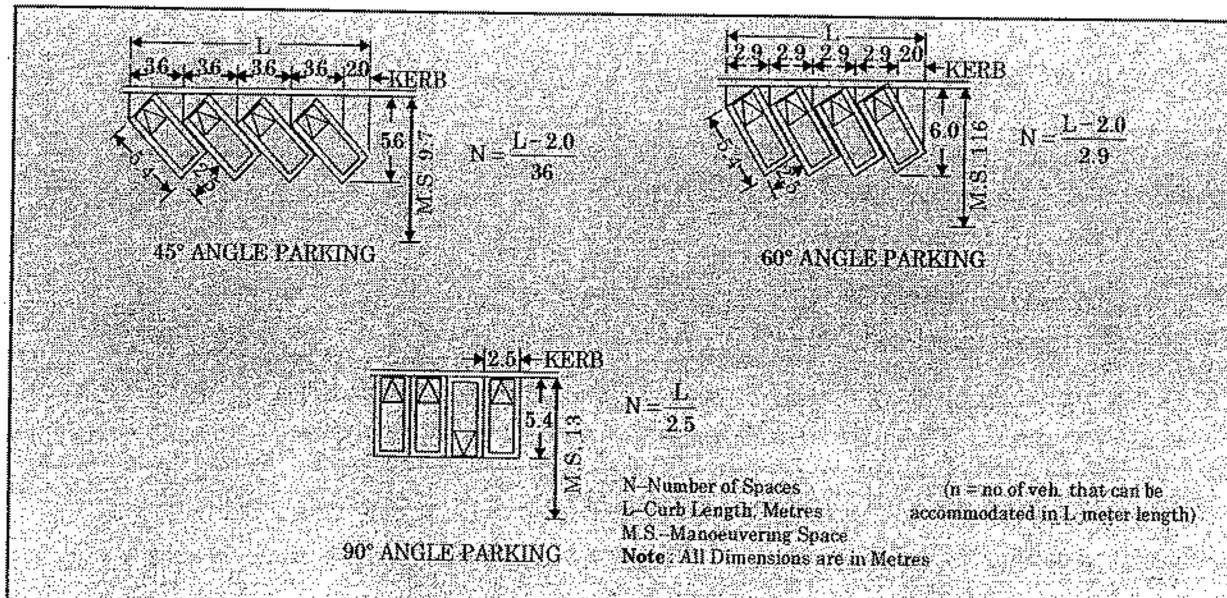


Fig. Various Patterns of Kerb Parking

Off-Stree Parking

- When parking facility is provided at a separate place away from the kerb, it is known as off-street parking. The main advantage of this method is that there is no undue congestion and delay on the road as in kerb parking.
- Parking studies are covered out using video recording.

ACCIDENT STUDIES

- Accident studies are used to find out the reason and cause behind accident and to take preventive measures in term of design control.

The various records that are maintained in accident studies are :

(a) **Location files :** Useful in identification of points of high accidents.

(b) **Spot Maps :** Accident location spot maps show accidents by spots, pins or symbols or maps.

(c) **Condition diagram :** A drawing is prepared to the scale showing all the important physical condition of an accident to be studied.

(d) **Collision diagram :** These diagram showing the approximate path of vehicles and pedestrians involved in the accidents.

ANALYSIS OF INDIVIDUAL TRAFFIC ACCIDENTS

- Some of the typical modes of vehicular accidents are:

(i) A moving vehicle collides with a parked vehicle

(ii) Two vehicles approaching from different directions collide at a intersection

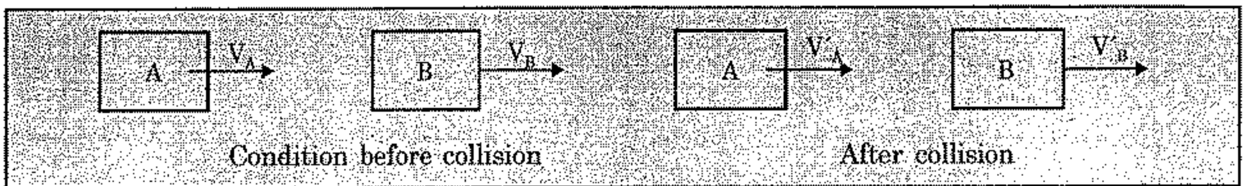
(iii) Head-on collision of two vehicles approaching from opposite directions

(iv) A moving vehicle collides with a stationary object like an electric pole, tree or a rigid structure.

For analysis purpose we compute the initial velocity involved in the accident. In the analysis following assumptions are made

1. When skid marks are present, it is assumed than 100% skid occurs and if marks are not prsent free collision is assumed.

2. Coefficient of restitution = $\dfrac{\text{Velocity Separation}}{\text{Velocity of approach}} = \dfrac{V_B{}' - V_A{}'}{V_A - V_B} = e$



for perfectly elastic collision e = 1

for perfectly plastic collision e = 0

- When vehicles are on the same path, collision is assumed to be a plastic one i.e. after collision two vehicles will move together.

$$e = 0,\ V_B{}' = V_A{}'$$

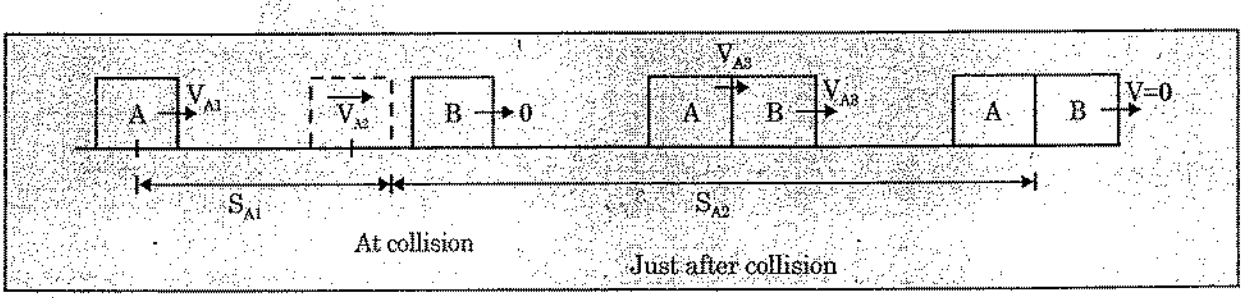
$$\boxed{\text{A}\ \text{B} \Rightarrow V'_A = V'_B \text{ after collision}}$$

Equation of momentum in this case is given by

$$\boxed{m_A V_A + m_B V_B = (m_A + m_B) V'}$$

Analysis

Case I : Collision of moving vehicles with parked vehicle along the line.



	Vehicle A	Vehicle B
Intial speed	VA_1	0
Just before collision	VA_2	0
Just after collision	VA_3	VA_3
Final speed	0	0

- Skid mark length between V_{A1} & V_{A2} = S_{A1}.
- Skid mark length from <u>after collision</u> to <u>stop condition</u> = S_{A2}.
- Our Aim is to calculate V_{A1}, when coefficient of friction (μ) is given.

We know that $V^2 = u^2 + 2as$ [a = acceleration; s = distance travelled]

$$\Rightarrow \quad V_{A2}^2 = V_{A1}^2 - 2(\mu g)(SA_1) \text{ [decleration is given by } (ma = \mu_1 mg)] \quad \text{............ (i)}$$

Also,

$$0 = V_{A3}^2 - 2\mu g(SA_2) \text{ [From after collision stage]........ (ii)}$$

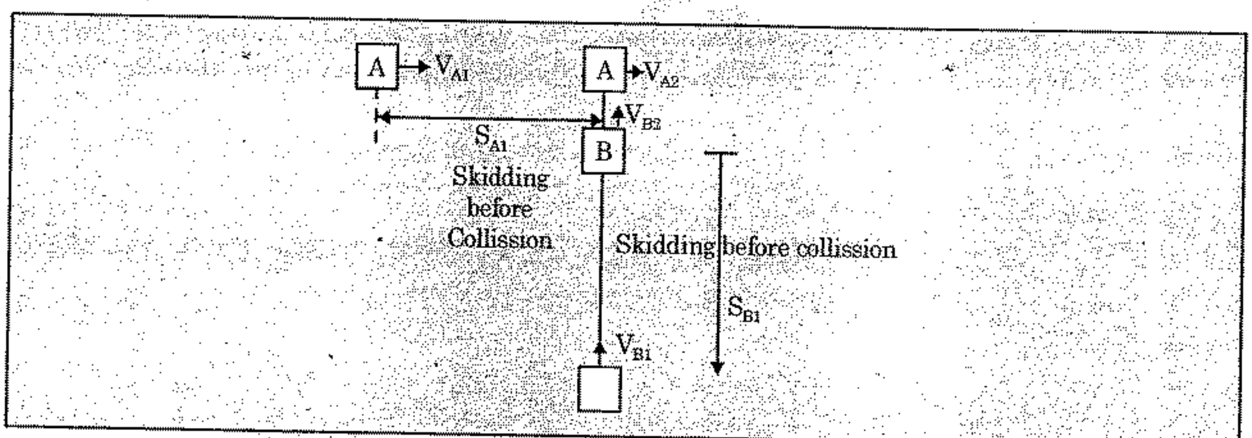
Now since,

$$m_A VA_2 + mB \times 0 = (m_A + m_B)VA_3$$

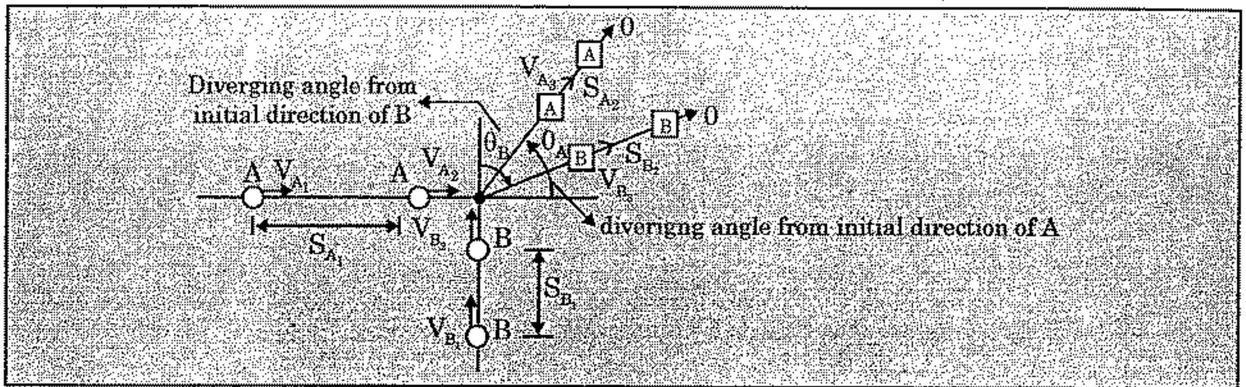
$$V_{A2} = \frac{m_A + m_B}{m_A} V_{A3} \quad \text{............ (iii)}$$

From (ii) we calculate V_{A_3} then we obtain V_{A_2} from (iii). Finally V_{A1}, is calculated from (i).

Case II : Collision of vehicles moving at right analysis :



- Two vehicles A & B are moving at right angles to each other and collides.
- Let A vehicle is initially moving with velocity of V_{A_1} and just before collision its velocity is V_{A_2} due to reduction in speed on account of brake application to avoid collision.
- Similarly, vehicle B is moving with velocity of V_{B_1} initially and just before collision its velocity is V_{B_2}.
- S_{A_1} is the skid distance due to brake application of vehicle A and S_{B_1} is the skid distance due to brake application of veh. B.
- After collision, Veh. A deviates from it, original path by angle θ_A and starts moving with velocity VA_3, where as, vehicle B deviates from its original path by angle θ_B and starts moving with velocity VB_3.
- Vehicle A stops in a distance of SA_2 after collision and vehicle B stops in a distance of SB_2 after collision.



Our Aim is to calculate V_{A_1} **&** V_{B_1}

$$V_{A_3}^2 = 2\mu g S_{A_2} \quad \text{........(i)}$$

$$V_{B_3}^2 = 2\mu g S_{B_2} \quad \text{........(ii)}$$

Also

$$V_{A_2}^2 = V_{A_1}^2 - 2\mu g S_{A_1} \quad \text{........(iii)}$$

$$V_{B_2}^2 = V_{B_1}^2 - 2g\mu S_{B_1} \quad \text{........(iv)}$$

From Momentum Conservation in X-direction

$$m_A V_{A2} = m_A V_{A_3} \cos\theta_A + m_B \sin\theta_B V_{B_3} \quad \text{.......... (v)}$$

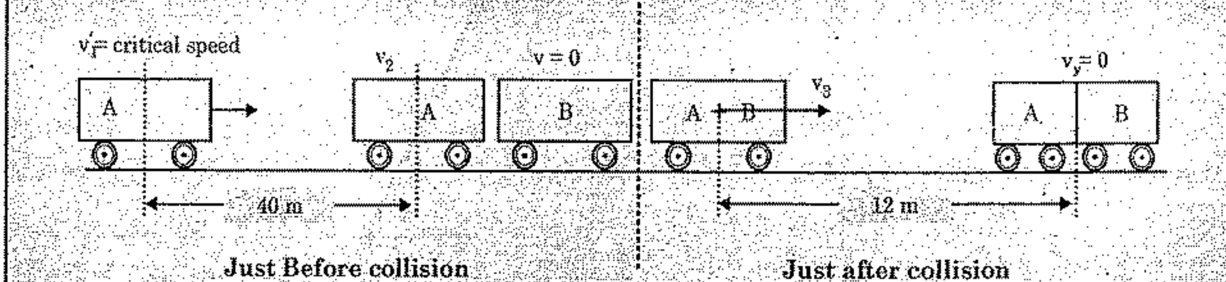
$$m_B V_{B2} = m_A V_{A_3} \sin\theta_A + m_B V_{B_3} \cos\theta_B \quad \text{.......... (vi)}$$

From these equation solutions can be obtained i.e V_{A_1} and V_{B_1} can be foundout.

Example 15

A Vehicle applies brakes and skids through a distance 40 m, before colliding with another parked vehicle, the weight of which is 60% of former. From fundamental principal. Compute the critical speed of moving vehicle. If distance travelled by both vehicle after collision is 12 m before stopping f = 0.60.

Sol.



Assumption

1. Collision is perfectly plastic 2. Brake efficiency = 100%

Steps :

1. After collision for (A) + (B)

$$\Rightarrow \quad v_3^2 = 2gfs_2$$

$$v_3 = \sqrt{2gfs_2} = \sqrt{2 \times 9.81 \times 0.60 \times 12} = 11.88\,\text{m/sec}$$

2. Momentum Equation

Just before collision = just after collision

$$\Rightarrow \quad m_A \times v_2 + m_B \times 0 = (m_A + m_B) \times v_3$$

$$\therefore \quad v_2 = \frac{(m_A + m_B)}{m_A} \times v_3$$

$$= \frac{(m_A + 0.60\, m_A)}{m_A} \times v_3$$

$$v_2 = 1.60\, v_3$$

$$v_2 = 1.60 \times 11.88 = 19.017\,\text{m/sec}$$

3. Before Collision For Vehicle (A)

$$v_1^2 - v_2^2 = 2gfs_1$$

$$\Rightarrow \quad v_1 = \sqrt{v_2^2 + 2gfs_1}$$

$$= \sqrt{(19.017)^2 + (2 \times 9.81 \times 0.60 \times 40)} = 28.85\,\text{m/sec}$$

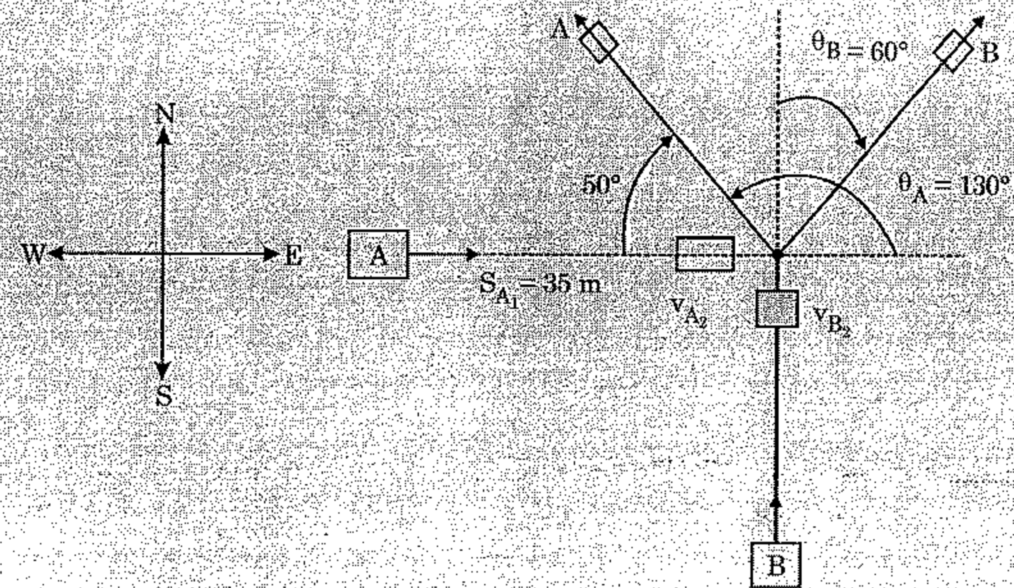
Example 16

Two Vehicles approaching at right angle A from West and B from South, collide with each other. After collision Vehicle A skids in the direction 50°N of West. B skids 60°E of N.

	(A)	(B)
Initial Skid Distance	35 m	20 m
Skid Distance After Collision	15 m	30 m
Weight	0.75 of (B)	6t
f - coefficient of friction	0.55	

Find out the initial speed of the two Vehicles.

Sol.



1. **After Collision for Vehicle (A)**

$$v_{A_3} = \sqrt{2gfS_{A_2}} = \sqrt{(2\times9.81\times0.55\times15)} = 12.72 \text{ m/sec} \quad \text{... (i)}$$

For Vehicle (B)

$$v_{B_3} = \sqrt{2gfS_{B_2}} = \sqrt{(2\times9.81\times0.55\times36)} = 19.71 \text{ m/sec}$$

2. **From Momentum Equation**

In the Direction of Vehicle A [East]

$$\Rightarrow \quad m_A v_{A_2} + 0 = (m_A v_{A_3} \cos\theta_A + m_B v_{B_3} \sin\theta_B)$$

$$\therefore \quad v_{A_2} = \left(v_{A_3} \cos\theta_A + \frac{m_B}{m_A} v_{B_3} \sin\theta_B\right)$$

$$= 12.72\times\cos130° + \frac{m_B}{0.75\,m_B}\times19.71\times\sin60 = 14.58 \text{ m/sec}$$

Similarily in the direction of Vehicle B

$$\left(m_A\times0 + m_B\times v_{B_2}\right) = \left(m_A v_{A_3} \sin\theta_A + m_B v_{B_3} \cos\theta_B\right)$$

$$\therefore \quad v_{B_2} = \left(\frac{m_A}{m_B}\right) v_{A_3} \sin\theta_A + v_{B_3} \cos\theta_B = \left(0.75\times12.72\times\sin130° + 19.70\times\cos60°\right)$$

$$v_{B_2} = 17.15 \text{ m/sec}$$

3. **Before Collision For Vehicle (A)**

$$v_{A_1}^2 - v_{A_2}^2 = \left(2gfS_{A_1}\right)$$

$$\therefore \quad v_{A_1} = \sqrt{\left(v_{A_2}^2 + 2gfS_{A_1}\right)} = \sqrt{(14.57)^2 + \left(2\times9.81\times0.55\times35\right)} = 24.29 \text{ m/sec}$$

For vehicle (B)

$$v_{B_1}^2 - v_{B_2}^2 = 2gfS_{B_1}$$

$$v_{B_1} = \sqrt{v_{B_2}^2 + 2gfS_{B_1}} = \sqrt{(17.15)^2 + (2\times9.81\times0.55\times20)} = 22.59 \text{m/sec}$$

INTERSECTION

- Intersection is the area where two or more roads join or cross.
- At traffic intersection change in direction of movements may occur.
- Due to movement of traffic at intersection various types of conflicts occur like crossing conflict, merging conflict and diverging conflict.
- The movement of one vehicle can conflict with the movements of other vehicles in the same stream, the cross-streams, the opposing stream, and pedestrians in crosswalks.
- Consider a typical four-legged intersection as shown in figure.
- The number of conflicts for competing through movements are 4, while competing right turn and through movements are 8.
- The conflicts between right turn traffics are 4, and between left turn and merging traffic is 4.
- The conflicts created by pedestrians will be 8 taking into account all the four approaches.
- Diverging traffic also produces about 4 conflicts.
- Therefore, a typical four legged intersection has about 32 different types of conflicts of which 24 are vehicular conflict and 8 are Pedestrian conflict.

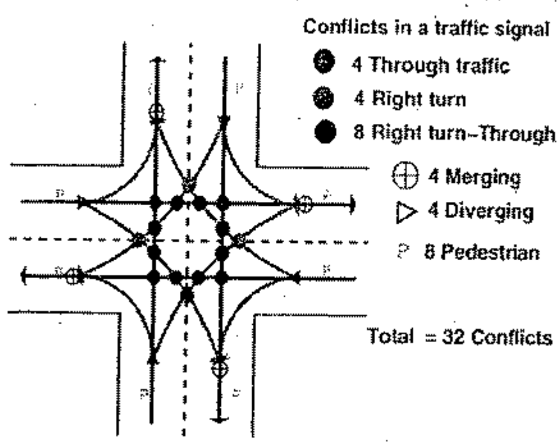
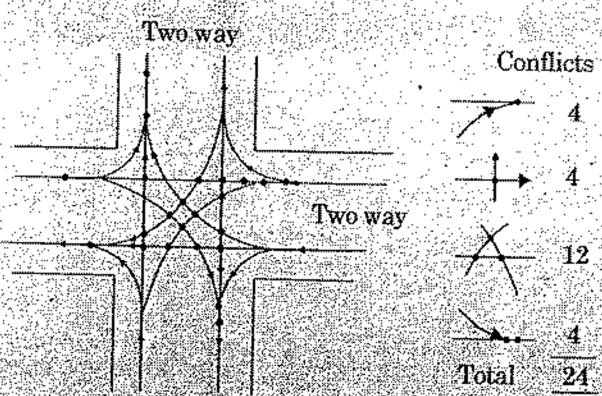
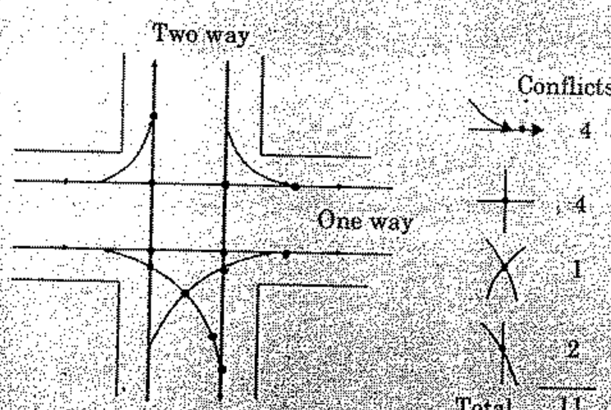


Figure Conflicts at an intersection



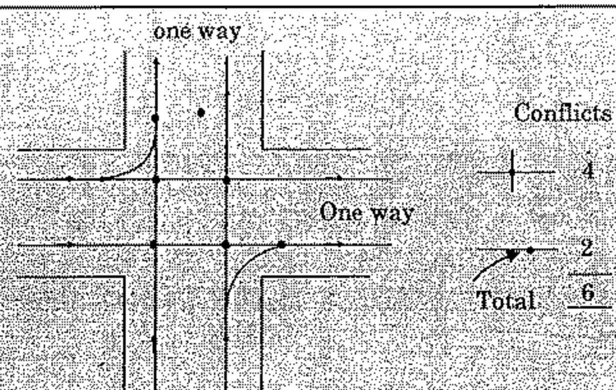
Vehicular Conflicts with Two-way traffic

Major Conflict = 4+12 = 16



Vehicular Conflicts with One-way Regulation on One Road

Major Conflict = 4 + 1 + 2 = 7



Vehicular Conflicts with one-way regulation on both Roads
Major conflict = 4

Number of lanes		Number of potential conflicts		
Road A	Road B	Both roads Two-way	A–one way B–Two way	Both roads One-way
2	2	24	11	6
2	3	24	11	8
2	4	32	17	10
3	3	24	13	11
4	4	44	25	18

- Crossing conflicts are the major conflicts and merging & diverging conflicts are minor conflicts.
- To reduce conflict we resort to intersection control.

  Intersection controls are categorised as

  (a) Passive control

  (b) Semi control

  (c) Active control

Passive Control :

- When volume of traffic is less no explict control is required.
- Road uses are required to follow traffic rules.
- Road sign and road markings are used in this case for example, GIVE WAY control requires the traffic on minor road to slow down and allow that on major road to proceed.

Semi Control :

- Drives are gently guided to avoid conflict.
- Channelization & traffic rotaries come under this category.

Active Control :

- Road uses are forced to follow the path suggested by traffic control agency.
- Traffic signals and grade separated intersection come under this category.

TYPES OF INTERSECTION

1. At-grade intersection 2.Grade-separated intersection

At-grade Intersections :

- All road intersection which meet at about the same level allowing traffic manoeuvres like merging, diverging, crossing, and weaving are called intersections at grade. These intersections may be further classified as unchannelized, channelized and rotary intersection.

Traffic Islands

- Traffic islands are raised areas constructed within the roadway to establish physical channels through which the vehicular traffic may be guided. Traffic islands often serve more than one function.

The traffic islands may be classified based on the function as :

(i) Divisional islands (ii) Channelizing islands
(iii) Pedestrian loading islands (iv) Rotary

- Divisional islands are intended to separate opposing flow of traffic on a highway with four or more lanes.
- By thus dividing the highway into two one-way roadways, the head on collisions are eliminated and in general other accidents are also reduced.
- The width of the divisional islands should be large if the head light glare is to be reduced during night driving. The kerb should be high enough to prevent vehicles from entering into the islands.
- Channelizing islands are used to guide the traffic into proper channel through the intersection area. Channelizing islands are very useful as traffic control devices for intersection at grade, particularly when the area is large.

The various uses of properly designed channelizing islands are listed below :

(i) The area of possible conflicts between traffic stream is reduced. By introducing channelizing islands, both the major and minor conflict areas are reduced.
(ii) They establish the desired angles of crossing and merging of traffic streams.
(iii) They are useful when the direction of the flow is to be changed.
(iv) They serve as convenient locations for other traffic control devices.
(v) They serve as refuge islands for pedestrians.

- Pedestrian loading islands are provided at regular bus stops and similar places for the protection of passengers. A pedestrian island at or near a cross walk to aid and protect pedestrian crossing the carriageway may be termed as pedestrian refuge islands. For crossing multilane highways, pedestrian refuge island after two or three lanes would be desirable.
- Rotary island is the large central island of a rotary intersection, this island is much larger than the central island of channelized intersection. The crossing manoeuvre is converted to weaving by providing sufficient weaving length.

Grade Separated Interaction :

- Intersection at grade is eliminated by the use of grade-separation structures that permit the cross flow of traffic at different levels without interruptions.

- A grade separation is a crossing of two highways, a highway and a railroad, or a pedestrian walkway, and a highway at different levels.
- An overpass is a highway passing over an intersection street, railroad, or pedestrian facility.
- An underpass is a highway passing under an intersecting street, railroad or pedestrian facility.

Interchange

An interchange is a grade separated intersection with connecting roadways (ramps) for turning traffic between highway approaches.

- Interchange will be necessary at all crossings of a highway which is to be developed to completely access controlled standard. Similarly, interchanges will also be required at all major crossings on highways developed to expressway standard.
- An interchange may be justified when an at grade inter-section fails to handle the volume of traffice resulting in serious congrestion and frequent choking of the inter-section. This situation may arise when the total traffice of all the arms of the intersection is in excess of 10,000 pcu's per hour.

Types of Interchanges

- Interchanges are generally described by the pattern of the various turning roadways or ramps which determine their geometric configuration. The ramps can be broadly classified into the following four basic types also illustrated in fig.

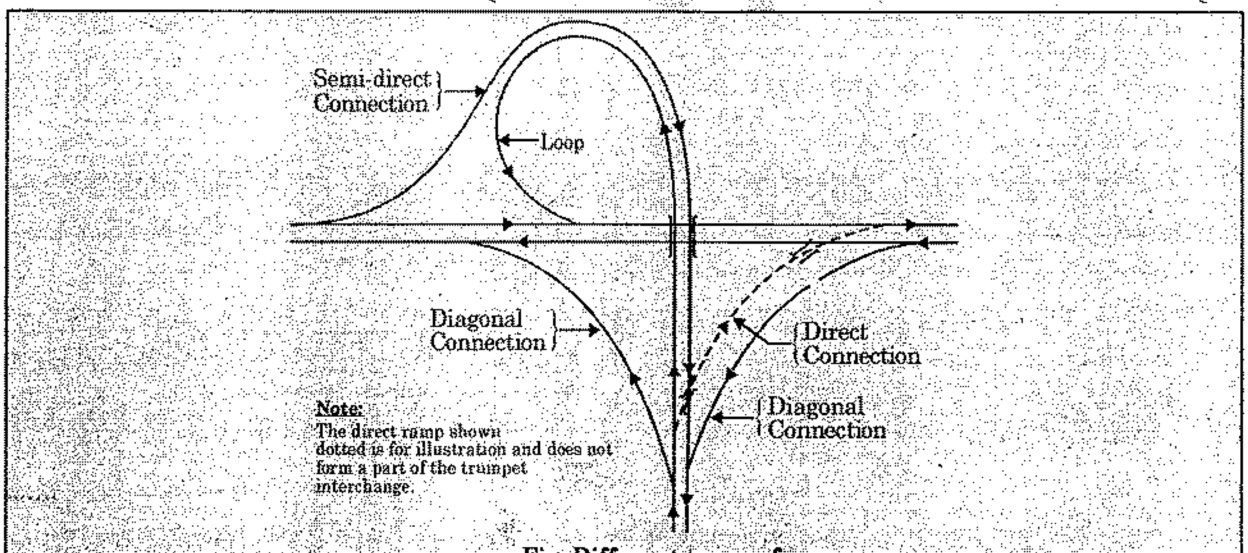


Fig. Different types of ramps

(i) Left turning roadways referred to as diagonal ramp or outer connection depending on its shape or type of interchange.

(ii) A loop which is a ramp for right turns accomplished by a left exit and turn to the left through about 270°.

(iii) Semi-direct connection which is a ramp for right turns accomplished through a partial deviation from the intended path.

(iv) Direct connection which is a ramp for right turns accomplished by a right directional and natural manoeuvre involving least extra travel distance.

- The common geometric configuration of interchanges are the trumpet, diamond, cloverleaf, rotary and directional.

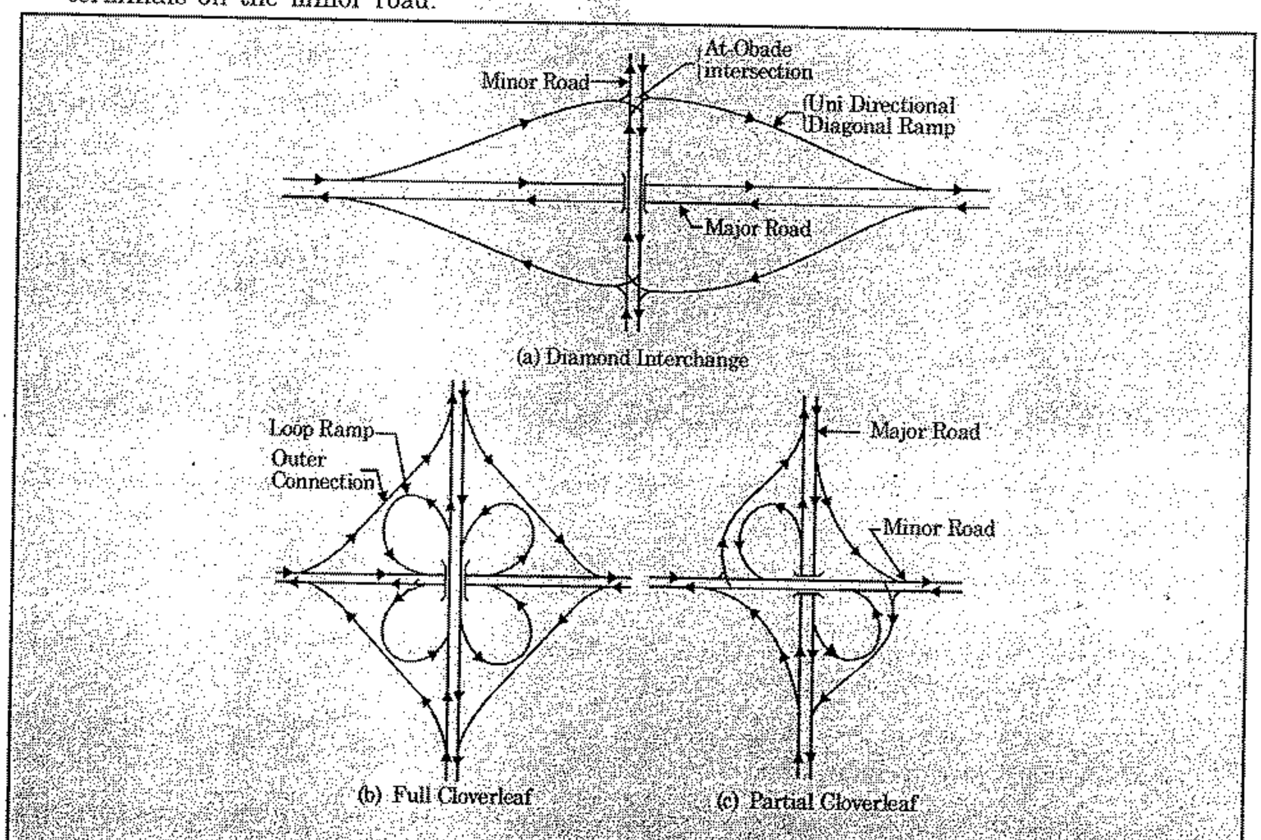
Trumpet Interchange

Fig. shows a typical 3-leg interchange which takes the shape of trumpet. This is the simplest interchange form adaptable to 'T' or 'Y' intersections. Of the two right turning movements one is negotiated by a loop while the other is by a semi-direct connection.

- Diagonal ramps are provided for left turning movements.
- The ramps catering for heavy traffice volumes should preferably be provided with direct connections. Fig. 1, illustrates the replacment of a loop ramp by a direct connection.

Diamond Interchange

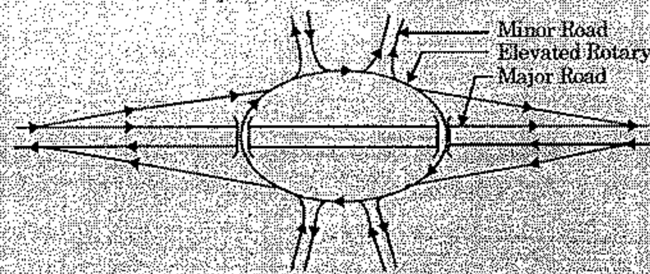
- Fig. 2(a) shows a typical diamond interchange. Diamond interchange is the simplest of 4-leg interchange designs.
- The ramps which provide for one way movement are usually elongated along the major highway and may be curved parallel to the major highway.
- The ramp terminals on the minor road are at-grade intersections providing for right and left turnings movements.
- These at-grade intersections may be controlled by signals if warranted by traffice volumes or in the absence of adequate sight distance.
- The diamond design requires minimum land, involves only a small extra travel distance for right turning traffic, is the least costly, and will be found ideal for most of the cases both in urban and rural areas.
- However, this type of interchange has the demerit of limited capacity because of the at-grade terminals on the minor road.



(a) Diamond Interchange

(b) Full Cloverleaf

(c) Partial Cloverleaf



(d) Rotary Interchange

Cloverleaf Interchange

- Fig. 2(b), shows a typical cloverleaf interchange. The design consists of one loop ramp for right turning traffic and one outer connection for left turning traffice in each quadrant.
- Vehicles desiring to turn right are required to turn left through about 270 degrees before attaining the desired direction.
- This type of interchange provides for continuous movement to all interchanging traffic and is particularly suitable for the crossing of two major roads of equal importance in rural areas.
- In urban areas, this type of interchange tends to use up too much of costly urban space.
- Cloverleaf design involves appreciable extra travel distance for the right moving traffice and requires a large space.
- Though all crossing movement conflicts are eliminated, a weaving section is created between the exit and entry points near the structure along each direction of travel on the intersecting roads.
- In cases where at-grade corssing on one of the roads can be tolerated, full cloverleaf development will not be required.
- For such cases, partial cloverleaf which is a modification that combines some elements of a diamond interchange with one or more loops to eliminate only the more critical conflicts can be adopted.

Rotary Interchange

- This type of design is particularly useful where a number of roads intersect at the interchange and in locations where sufficient land is available.
- It requires the construction of two bridges and generally necessitates more land than for a diamond layout.
- The main highway goes over or under the rotary intersection and turning movements are accommodated by the diagonal ramps.

Directional Interchange

- Directional interchanges have ramps for right turning traffic which follow the natural direction of movement.
- This type of design requires more than one structure or a 3-level structure.
- Though operationally more efficient than other designs, these generally turn out to be very expensive.

TRAFFIC ROTARIES

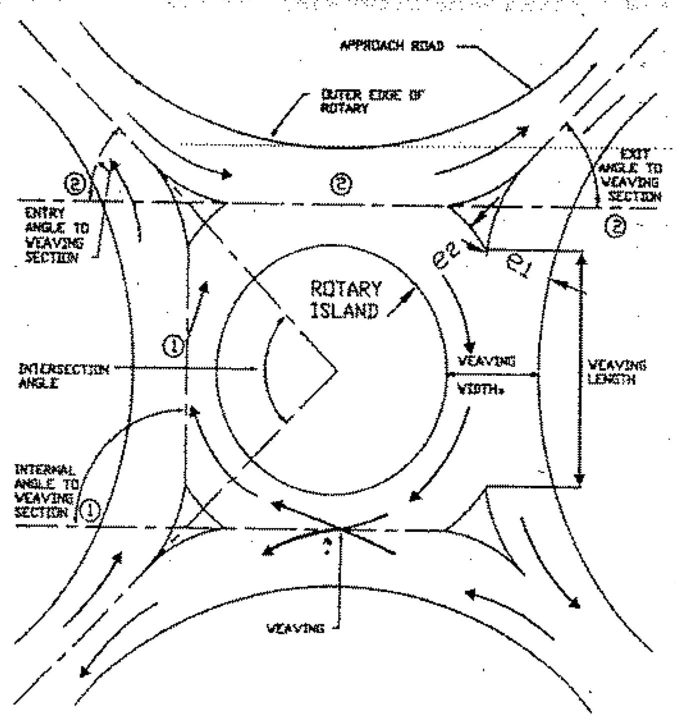
- A traffic rotary is a specialized form of "at-grade" intersection where vehicles from the converging arms are forced to move round an island in one direction in an orderly and regimented manner and "weave" out of the rotary movement into their desired direction.
- Traffic rotaries reduces the complexity of crossing traffic by forcing them into weaving operations.
- The shape and size of the rotary are determined by the traffic volume and share of turning movements. Capacity assessment of a rotary is done by analyzing the section having the greatest proportion of weaving traffic.

IMPORTANT DEFINITIONS ABOUT ROTARY ELEMENT

(1) **At-grade intersection :** An intersection where all roadways join or cross at the same level.

(2) **Diverging :** The dividing of a single stream of traffic into separate streams.

(3) **Intersection angle :** The angle between two intersection legs.



e_1 = Width at entry
e_2 = Width of Non Weaving Section

Rotary elements

(4) **Merging :** The converging of separate streams of traffic into a single stream.

(5) **Rotary Intersection :** A road junction laid out for movement of traffic in one direction round a central island.

(6) **Rotary Island :** A traffic island located in the centre of an intersection to compel movement in a clock-wise direction and thus substitute weaving of traffic around the island instead of direct crossing or vehicle pathways.

(7) **Weaving** : The combined movement of merging and diverging of traffic streams moving in the same general direction.

(8) **Weaving length** : The length of a section of a rotary in which weaving occurs.

ADVANTAGE AND DISADVANTAGE OF ROTARY

Advantage

The advantages of traffic rotaries are:

(a) All traffic proceeds at a fairly uniform speed. Frequent stopping and starting are avoided. Actually vehicle do not stopped rotary.

(b) Weaving replaces the usual crossing movements. Direct conflict is eliminated, all traffic streams merging of diverging at small angles. Accidents occurring from such movements are usually of a minor nature.

(c) Rotaries are especially suited for intersections with five or more intersection legs though these can also be adopted at intersections with 3 or 4 legs.

(d) Rotaries are self governing.

Disadvantages

(a) All vehicles are forced to reduce speed even if traffic Volume is less.

(b) A rotary requires a comparatively larger area and may not be feasible in many built-up locations.

(c) Not suitable for padestrain movement because Vehicle do not stop.

(d) Where the angle of intersection between two roads is too acute, it becomes difficult to provide adequate weaving length.

(e) Traffic turning right has to travel a little extra distance.

(f) Because of the above limitation rotaries are not suitable for every location.

GUIDELINES FOR THE SELECTION OR ROTARIES

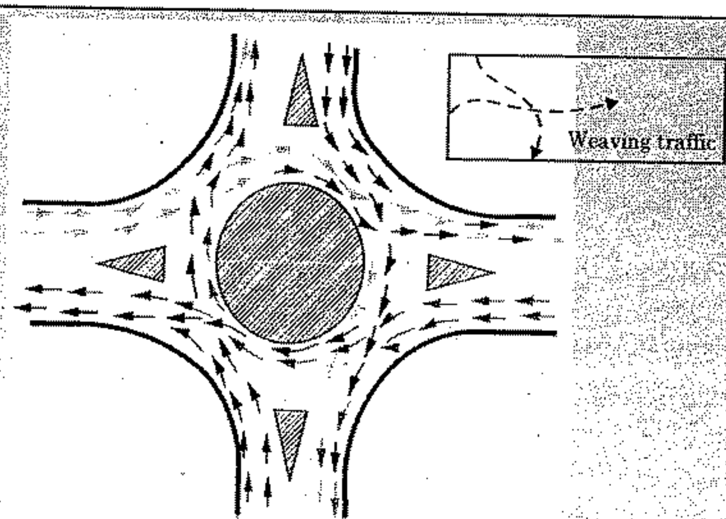
- There are few guidelines that help in deciding the suitability of a rotary. They are listed below.
  1. Rotaries are suitable when the traffic entering from all the four approaches are relatively equal.
  2. A total volume of about 3000 vehicles per hour can be considered as the upper limiting case and a volume of 500 vehicles per hour is the lower limit from all legs.
  3. A rotary is very beneficial when the proportion of the right-turn traffic is very high; typically if it is more than 30 percent.
  4. Rotaries are suitable when there are more than four approaches or if there is no separate lanes available for right-turn traffic. Rotaries are ideally suited if the intersection geometry is complex.

Traffic operations in a rotary

There are three traffic operation at rotary

1. **Diverging** : It is a traffic operation when the vehicles moving in one direction is separated into different streams according to their destinations.

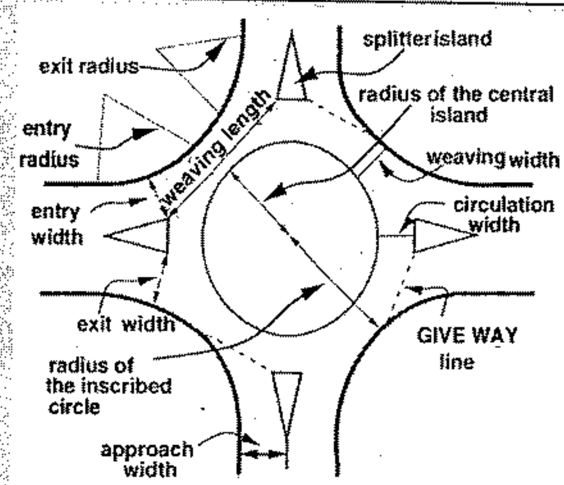
2. **Merging :** Merging is the opposite of diverging. Merging is referred to as the process of joining the traffic coming from different approaches and going to a common destination into a single stream.
3. **Weaving :** Weaving is the- combined movement of both merging and diverging movements in the same direction.



Traffic operations in a rotary

DESIGN ELEMENTS

- The design elements include design speed, radius at entry, exit and the central island, weaving length and width, entry and exit widths. In addition the capacity of the rotary can also be determined by using some empirical formula.



Design of rotary

Design speed

- All the vehicles are required to reduce their speed at a rotary. Therefore, the design speed of a rotary will be much lower than the roads leading to it.
- The normal practice is to keep the design speed as 30 and 40 kmph for urban and rural areas respectively.

Radii of curves at entry and exit

At entry

Entry to the rotary is not straight rather a curvataure is introduced. This will force the driver to reduce speed.

Rotary Design Speed V (kmph)	Suggested Values of Radius at Entry (metres)
40* 30**	20—35 15—25

* Speed generally suitable for rotaries in rural areas.

** Speed generally suitable for rotaries in urban areas and other restricted locations.

At Exit

- The radii of the curves at exit should be larger than that of the central island and at entry so as to encourage the drivers to pick up speed and clear away from the rotary expeditiously.
- For this reason, the radius of the exit curves may be kept about $1\frac{1}{2}$ to 2 times the radius of the entry curves.
- If, however, there is a large pedestrian traffic across the exit road, radii similar to those at entrances should be provided to keep the exit speeds reasonably low.

Radius of Central Island

- Theoretically, the radius of the central island should be equal to the radius at entry.
- In practice, however, the radius of the central island is kept slightly larger than that of the curve at entry, this being an attempt to give a slight preference to the traffic already on the rotary and to slow down the approaching traffic.
- A value of 1.33 times the radius of entry curve is suggested as a general guideline for adoption.

Weaving Length

- The weaving length determines the ease with which the vehicles can manoeuvre through the weaving section and thus determines the capacity of the rotary.
- The weaving length is decided on the basis of factors such as the width of the weaving section, the average width of entry, total traffic and the proportion of weaving traffic in it.
- As a general rule, effort should be made to keep the weaving length at least 4 times the width of the weaving section.
- The following minimum values of weaving lengths for different design speeds should be observed.

Design Speed (kph)	Minimum Weaving Length (metres)
40 30	45 30

- In order to discourage speeding in the weaving sections, the maximum weaving length should be restricted to twice the values given above. Larger weaving length encourate ever speeding.

Width of Carriageway At Entry and Exit

- The carriageway width at entrance and exit of a rotary is governed by the amount of traffic entering and leaving the rotary. Entry width should be lower than the width of carriage way at approach.
- It is recommended that the minimum width of carriageway be at least 5 metre with necessary widening to account for the curvature of the road.

Carriageway width of the approach road	Radius at entry (m)	Width of carriageway at entry and exit (m)
7 m (2 lanes)	25—35	6.5
10.5 m (3 lanes)		7.0
14 m (4 lanes)		8.0
21 m (6 lanes)		13.0
7 m (2 lanes)	15–25	7.0
10.5 m (3 lanes)		7.5
14 m (4 lanes)		10.0
21 m (6 lanes)		15.0

Width of non-weaving section

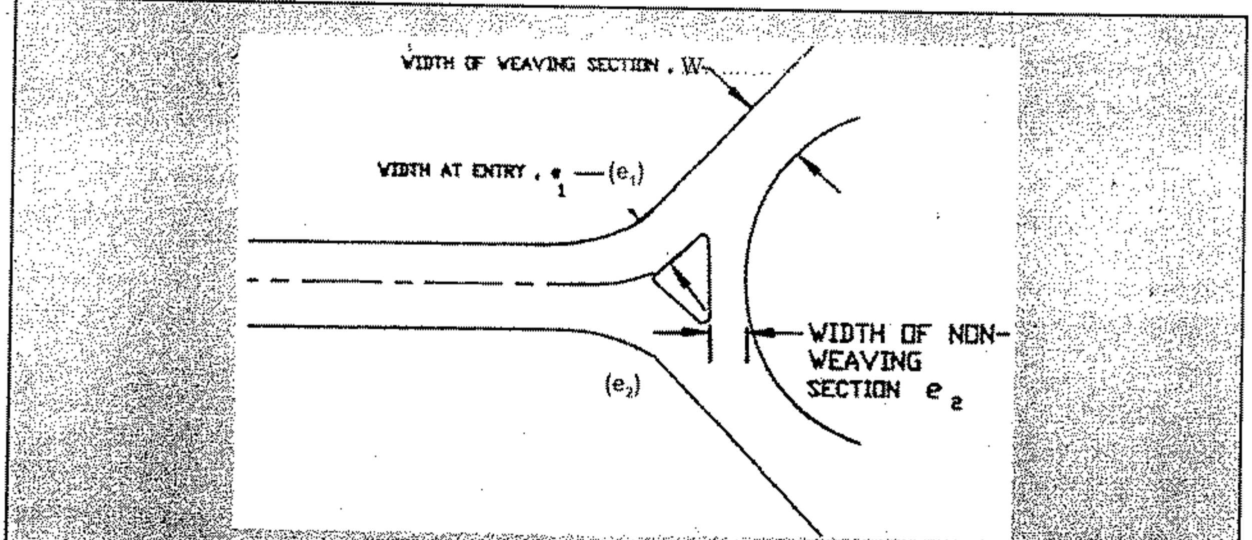
The width of non-weaving section of the rotary should be equal to the widest single entry into the rotary and should generally be less than the width of the weaving section.

Width of weaving section

The width of the weaving section of the rotary should be one traffic lane (3.5 m) wider than the mean entry width.

$$N_{weaving} = \frac{e_1 + e_2}{2} + 3.5$$

e_1 = Entry width (m) e_2 = exit width (m)



Width of rotary carriageway

Entry and Exit Angles

- Entry angles should be larger than exit angle, and it is desirable that the entry angles should be 60° if possible.
- The exit angles should be small, even tangential. An idealised design showing entry angles of 60° and exit angles of 30°.

Capacity of the rotary

- Capacity of rotary is determined by the capacity of each weaving section. The overall capacity of the rotary is reported as the min. value.
- Capacity of the individual weaving sections depends on factors such as (i) width of the weaving section (ii) average width of entry into the rotary (iii) the weaving length and (iv) proportion of weaving traffic and could be calculated from the following formula:

$$Q_v = \frac{280w\left(1+\frac{e}{w}\right)\left(1-\frac{p}{3}\right)}{1+\frac{w}{l}}$$

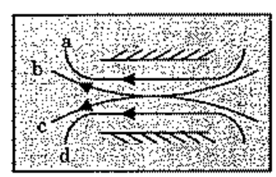
where,

Q_v = Practical capacity of the weaving section of the rotary in passenger car units (Pcu) per hour.

w = <u>width of weaving section in metres</u> (within the range of 6—18 m)

e = <u>average entry width in metres</u> (i.e, average of entry & exit width) $\frac{e}{w}$ to be within an range of 0.4 to 1.00

l = <u>length in metres of the weaving section</u> between the ends of channelising islands ($\frac{w}{l}$ to be within the range 0.12 and 0.4)



p = <u>proportion of weaving traffic</u>, i.e., ratio of sum of crossing streams to the total traffic on the weaving section ($p = \frac{b+c}{a+b+c+d}$, range of p being 0.4 to 1.0)

- p will be calculated at all the weaving section and height value is adopted. If e, w etc. are different for difference weaving section, we will have to calculate capacity of all the weaving section& minimum value will be adopted.

<u>The following adjustments in the capacity calculated by the above formula are suggested:</u>

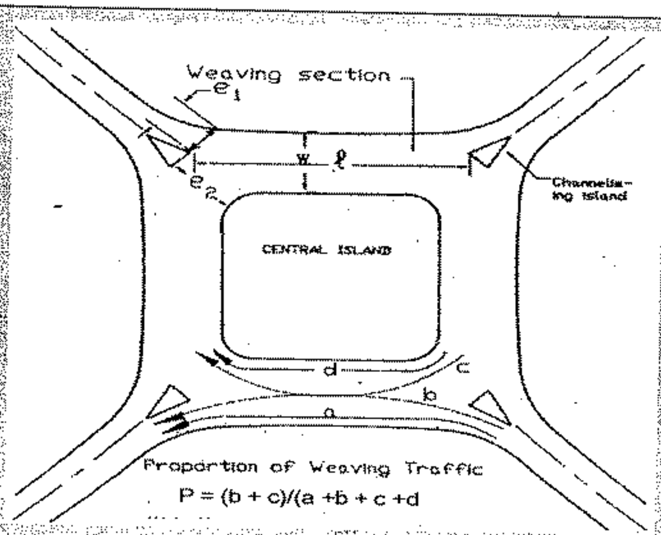
(i) Where the entry angle is between 0° and 15°, deduct 5 percent from the capacity of the weaving section

(ii) Where the entry angle is between 15° and 30°, deduct $2\frac{1}{2}$ percent from the capacity of the weaving section.

(iii) Where, the exit angle is between 60° and 75°, deduct $2\frac{1}{2}$ percent from the capacity of the weaving section.

(iv) Where the exit angle is greater than 75°, deduct 5 percent from the capacity of the weaving section.

(v) Where the internal angle is greater than 95°, deduct 5 percent from the capacity of the weaving section



Example 17

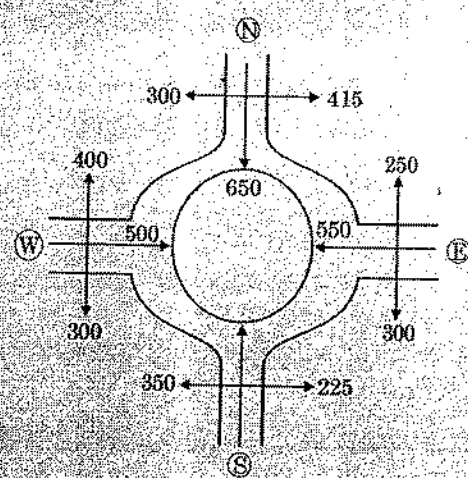
Traffic Flow in a urban area at right angle intersection of two major road are given as below. Both roads has a carriagewidth of 15 m.

Approach Road	Traffic Road (PCU / hr)		
	L.T.	*S.T.*	*R.T.*
North	415	650	300
East	300	550	250
South	350	400	225
West	400	500	300

Design and draw a rotary intersection and check for its practical capacity. Making suitable assumption as per IRC recommendation.

Sol. Practical capacity of rotary

$$Q_P = \frac{280\,W\left(1+\frac{e}{W}\right)\left(1-\frac{p}{3}\right)}{\left(1+\frac{W}{L}\right)} \quad \text{... (i)}$$



where W = Width of weaving section $= \left(\frac{e_1+e_2}{2}+3.5\right)$

$e_1 = 8.0\,\text{m(adopted)}$; $e_2 = 8.0\,\text{m(adopted)}$

$$\therefore \quad W = \left(\frac{e_1+e_2}{2}\right)+3.5 = \left(\frac{8+8}{2}\right)+3.5 = 11.5$$

L = length of weaving section

= 4 × width of weaving section

= 4 × 11.5 = 46 m

$$p = \text{Weaving Ratio} = \frac{b+c}{(a+b+c+d)}$$

where b and c are weaving traffic and a, d are the non-weaving traffic

For North-East

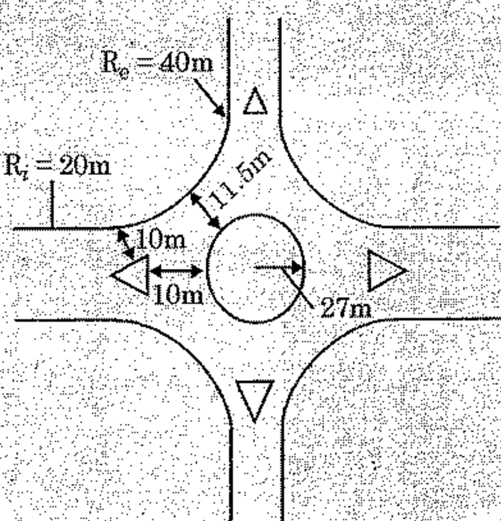
a = 415 ; b = (650 + 300) = 950 ; c = 500 + 225 = 725 ; d = 300

$$p = \frac{(950+725)}{(415+300+950+725)} = 0.70$$

For East-South

a = 300; b = (550 + 250) = 800 ; c = (650 + 300) = 950 ; d = 300

$$\text{Weaving ratio } p = \frac{(b+c)}{(a+b+c+d)} = \frac{(800+950)}{(800+950+300+300)} = 0.745$$



For South-West

a = 350 ; b = 400 + 225 = 625 ; c = (300 + 550) = 850 ; d = 250

$$p = \frac{b+c}{(a+b+c+d)} = \frac{(625+850)}{(350+625+800+250)} = 0.711$$

For West-North

a = 400; b = (300 + 500) = 800 ; c = (250 + 400) = 650, d = 225

$$p = \frac{b+c}{(a+b+c+d)} = \frac{(800+650)}{(400+800+650+225)} = 0.698$$

It is clear from equation (i), that the highest proportion of weaving traffic to non-weaving traffic will give the minimum capacity

$\therefore \quad p = 0.745$

Practical capacity $Q_P = \dfrac{280W\left(1+\dfrac{e}{W}\right)\left(1-\dfrac{p}{3}\right)}{\left(1+\dfrac{W}{L}\right)} = \dfrac{280\times 11.5\left(1+\dfrac{8.0}{11.5}\right)\left(1-\dfrac{0.745}{3}\right)}{\left(1+\dfrac{11.5}{46}\right)}$

$= 3283.28$ PCU/hr $= 3284$ PCU/hr

As per IRC

(i) The rotatory is located in an urban section and hence a design speed = 30 km/hr

(ii) Since the intersection legs carry almost equal traffic a round shaped central Island will be adopted

(iii) The entrance and exit angles will be 45°

(iv) Minimum radius of central Island will be 1.33 times the radius at entry

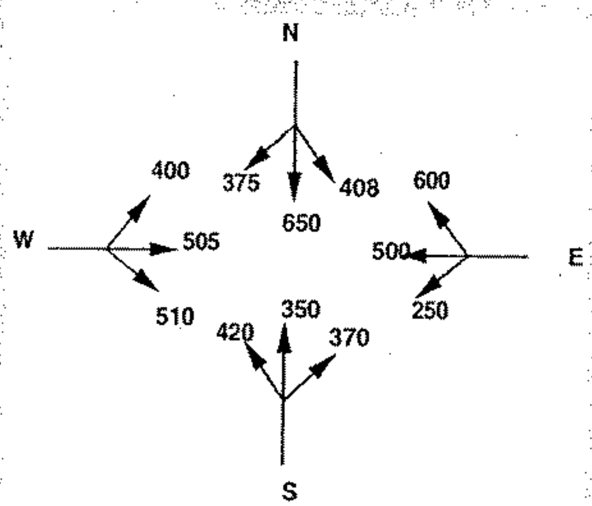
(v) Adopt radius at entry = 20 m

Radius at exit = 40 m

Hence, Radius of central Island = 1.33 × 20 = 27 m

Example 18

The width of a carriage way approaching an intersection is given as 15m. The entry and exit width at the rotary is 10 m. The traffic approaching the intersection from the four sides is shown in the figure below. Find the capacity of the rotary using the given data.



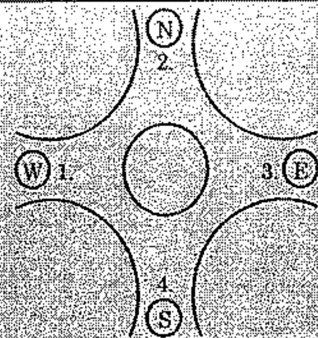
Sol. (i) The traffic from the four approaches negotiating through the roundabout is illustrated in figure.

(ii) Weaving width is calculated as, $w = \left[\dfrac{e_1+e_2}{2}\right] + 3.5 = 13.5$ m

(iii) Weaving length, l is calculated as $= 4 \times w = 54$ m

(iv) The proportion of weaving traffic to the non-weaving traffic in all the four approaches is found out first.

As it is a four legged rotary, hence the diagram will be as follows :



$V_{1-2} = 400$	$V_{23} = 408$	$V_{34} = 250$	$V_{41} = 420$
$V_{1-3} = 505$	$V_{24} = 650$	$V_{31} = 500$	$V_{42} = 350$
$V_{1-4} = 510$	$V_{21} = 375$	$V_{32} = 600$	$V_{43} = 370$

Weaving traffic in 1–2 section are the ones which are moving from

$1 \rightarrow 3$ i.e. V_{1-3}

$1 \rightarrow 4$ i.e. V_{1-4}

$4 \rightarrow 2$ i.e. V_{4-2}

$3 \rightarrow 2$ i.e. V_{3-2}

Total traffic at section 1 – 2 are the ones which are moving from

$1 \rightarrow 2$ i.e. V_{12}

$1 \rightarrow 3$ i.e. V_{13}

$1 \rightarrow 4$ i.e. V_{14}

$4 \rightarrow 2$ i.e. V_{42}

$4 \rightarrow 3$ i.e. V_{43}

$3 \rightarrow 2$ i.e. V_{32}

Hence $$p_{1-2} = \frac{V_{13} + V_{14} + V_{4-2} + V_{3-3}}{V_{1-2} + V_{1-3} + V_{1-4} + V_{4-2} + V_{4-3} + V_{32}} = \frac{1965}{2735} = 0.718$$

Similarly, $$p_{2-3} = \frac{V_{2-4} + V_{2-1} + V_{1-3} + V_{4-3}}{V_{2-3} + V_{2-4} + V_{2-1} + V_{1-3} + V_{1-4} + V_{4-3}} = 0.674$$

$$p_{3-4} = \frac{V_{3-1} + V_{3-2} + V_{2-4} + V_{1-4}}{V_{3-4} + V_{3-1} + V_{3-2} + V_{2-4} + V_{1-4} + V_{2-1}} = 0.783$$

$$p_{4-1} = \frac{V_{4-2} + V_{4-3} + V_{3-1} + V_{2-1}}{V_{4-1} + V_{4-2} + V_{4-3} + V_{3-2} + V_{3-1} + V_{2-1}} = 0.61$$

Hence largest value will be adopted for capcity determination i.e. p_{3-4}.

$\Rightarrow$ $$\text{Capacity} = \frac{280 \times 13.5 \left[1 + \frac{10}{13.5}\right]\left(1 - \frac{0.783}{3}\right)}{\left(1 + \frac{13.5}{54}\right)}$$

$$= 3890$$

Example 19

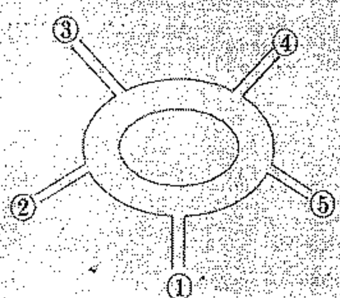
A road intersection has five legs designated as 1, 2, 3, 4 and 5. Leg 1 is in N-S direction and others are marked clockwise. The traffic Volumes in terms of PCU(V_{ij}) per hour during peak period are given below:

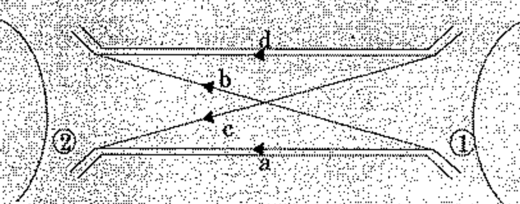
V_{12}	37	V_{31}	466	V_{41}	182	V_{51}	45
V_{13}	303	V_{32}	122	V_{42}	54	V_{52}	132
V_{14}	64	V_{34}	47	V_{43}	18	V_{53}	62
V_{15}	52	V_{35}	657	V_{45}	116	V_{54}	15

Find the weaving ratio between the legs 1 and 2.

What is the use of this value? Draw a sketch showing the traffic volumes between legs 1 and 2.

Sol.





For weaving ratio between (1) & (2)

$$a = v_{12} = 37$$

$$b = v_{13} + v_{14} + v_{15} = 303 + 64 + 52 = 419$$

$$c = v_{32} + v_{42} + v_{52} = 122 + 54 + 132 = 308$$

$$d = v_{43} + v_{53} + v_{54} = 47 + 62 + 15 = 124$$

$$p = \frac{b+c}{a+b+c+d} = \frac{419+308}{37+419+308+124} = 0.818$$

"Weaving ratio shows percentage of traffic between 1 & 2 legs that will be crossing each other w.r.t total traffic between two legs."

TRAFFIC CONTROL DEVICES

Traffic control devices are all the traffic signals, signs, pavement marking, or other devices placed or erected with the approval of a traffic authority having the necessary jurisdiction, to regulate, warn, or guide traffic.

Function of Traffic Control Devices

Devices are classified into three functional groups as follows :

1. Regulatory devices

- These give the road user notice of traffic laws or regulations that apply at a given place or on given roadway.
- Disregard of such devices in punishable as an infraction, violation, or misdemeanor. *e.g.,* stop,no turning, do not enter, no parking, one-way street, etc.

2. Warning devices

These call attention of the road user to conditions, on or adjacent to the roadway, that are potentially hazardous to traffic operations. *e.g.*, road narrow, divided highway ends, slippery when wet, railroad crossing, etc.

3. Guiding devices

These provide directions and information to the road user regarding route designations, distances, destinations point of interest, and other geographical or cultural information. *e.g.*, Airport 20 km., Railway 30 km.

TRAFFIC SIGNALS

Traffic signals are control devices which could alternately direct the traffic to stop and proceed at intersections using red and green traffic light signals automatically. Main requirements of traffic signal are to draw attention, provide meaning and time to respond & to have minimum waste of time.

Type of Traffic Signals

The signals are classified into the following types :

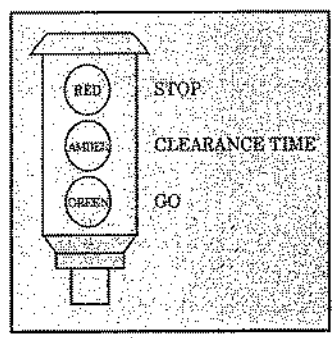
(i) Traffic control signals

(a) Fixed-time signal (b) Semi actuated signal (c) Fully activated signal

(ii) Pedestrian signal

(iii) Special traffic signal

- The traffic control signals have three coloured light glows facing each direction of traffic flow.
- The red light is meant for stop, the green light indicates Go and the amber or yellow light allows the clearance time for the vehicles which enter the intersection area by the end of green time, to clear off.



Fixed-time signal or pre-timed signals are set to repeat regularly a cycle of red, amber and green lights. The timing of each phase of the cycle is predetermined based on the traffic studies. Signal timing does not change in response to changes in traffic flow at the intensection. No vehicle detection is necessary with this mode of operation.

- The main draw back of the signal is that some times the traffic flow on one road may be almost nil and traffic on the cross road may be quite heavy but time available for cross road may be less.

Semi-actuated Signal is a signal whose timing (cycle length, green time, etc.) is affected when vehicles are detected (by video, pavement-embedded inductance loop detectors, etc.) on some, but not all, approaches.

- This mode of operation is usually found where a low-volume road intersects a high-volume road, often referred to as the minor and major street, respectively.
- In such cases, green time is allocated to the major street until vehicles are detected on the minor street : then the green indication is briefly allocated to the minor street and then returned to the major street.

Fully actuated signal is a signal whose timing (cycle length, green time, etc.) is completely influenced by the traffic volumes, when detected, on all of the approaches.

- Fully actuated signals are most commonly used at intersections of two major streets and where substantial variations exist in all approach traffic volumes over the course of a day.

Type of Co-ordination of Traffic Signal System

There are four general types of co-ordination of signals for road network, as listed below :

(i) Simultaneous system
(ii) Alternate system
(ii) Simple progressive system, and
(iv) Flexible progressive system

Simultaneous System

- In this system all the signals along a given road always show the same indication (green, red etc.) at the same time.
- As the division of cycle is also the same at all intersections, this system does not work satisfactorily.

Alternate System

- In this system, alternate signals or groups of signals show opposite indications in a route at the same time.
- This system is also operated by a single controller, but by reversing the red and green indicator connections at successive signal systems.
- This system generally is considered to be more satisfactory than the simultaneous system.

Simple Progressive System

- A time schedule is made to permit, as nearly as possible, a continuous operation of groups of vehicles along the main road at a reasonable speed.
- The signal phases controlling "Go" indications along this road is scheduled to work at the predetermined time schedule.
- The phases and intervals at each signal installation may be different; but each signal unit works as fixed time signal, with equal signal cycle length.

Flexible Progressive System

- In this system it is possible to automatically vary the length of cycle, cycle division and the time schedule at each signalized intersection with the help of a computer.
- This is the most efficient system of all the four types described above.

ANALYSIS OF TRAFFIC AT SIGNALISED INTERSECTION

Concept & definition :

Cycle : A signal cycle is one complete rotation through all the indications provided. (Red, Green, Yellow)

Cycle Length (C) : It is the time in seconds that it takes a signal to complete on full cycle of indication i.e. the time interval between the starting of green for one approach till next time the green starts.

Internal : Interval represents change from one stage to other. The two types of interval are

(a) Change interval (b) Clearance interval

- Change interval represents yellow time. It indicates the interval between the green and red signal. Clearance interval represents All red time. It is included after each yellow interval indicating a period during which all signal faces show red and is used for clearing off the vehicles in the intersection.

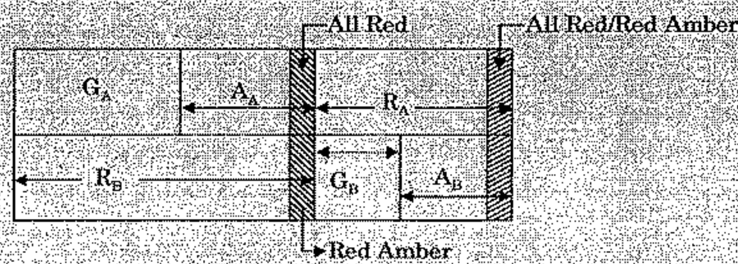


Fig. : Two phase signal system

$$R_B = G_A + A_A$$

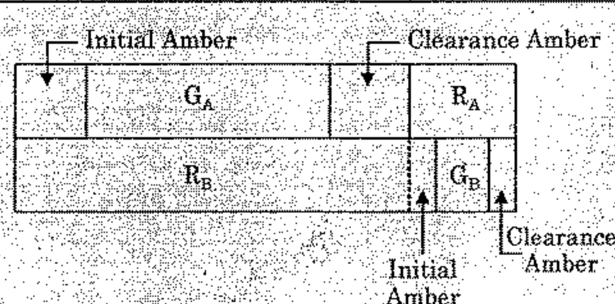
$$R_A = G_B + A_B$$

Where A_A = Yellow time or Amber time for Road A.

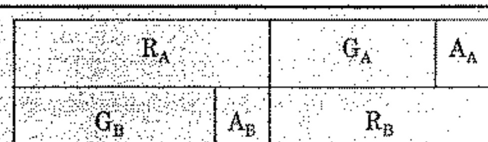
G_A = Green time for road A

R_A = Red time for road A.

Some times the arrangement of traffic signal timing is also as show below. Initial amber is for get set to Go. Vehicles are not allowed to move during red-amber (also called initial amber)



yet another arrangement can be



- Yellow time is provided after green time for movement. The purpose is to warn a driver approaching the intersection during the end of green time about the coming of a red signal.
- The design consideration is that a driver approaching the intersection with design speed should be able to stop at the stop line of the intersection before the start of red time.
- It can be approximately calculated as SSD. Where SSD is the stopping sight distance & V is the design speed.

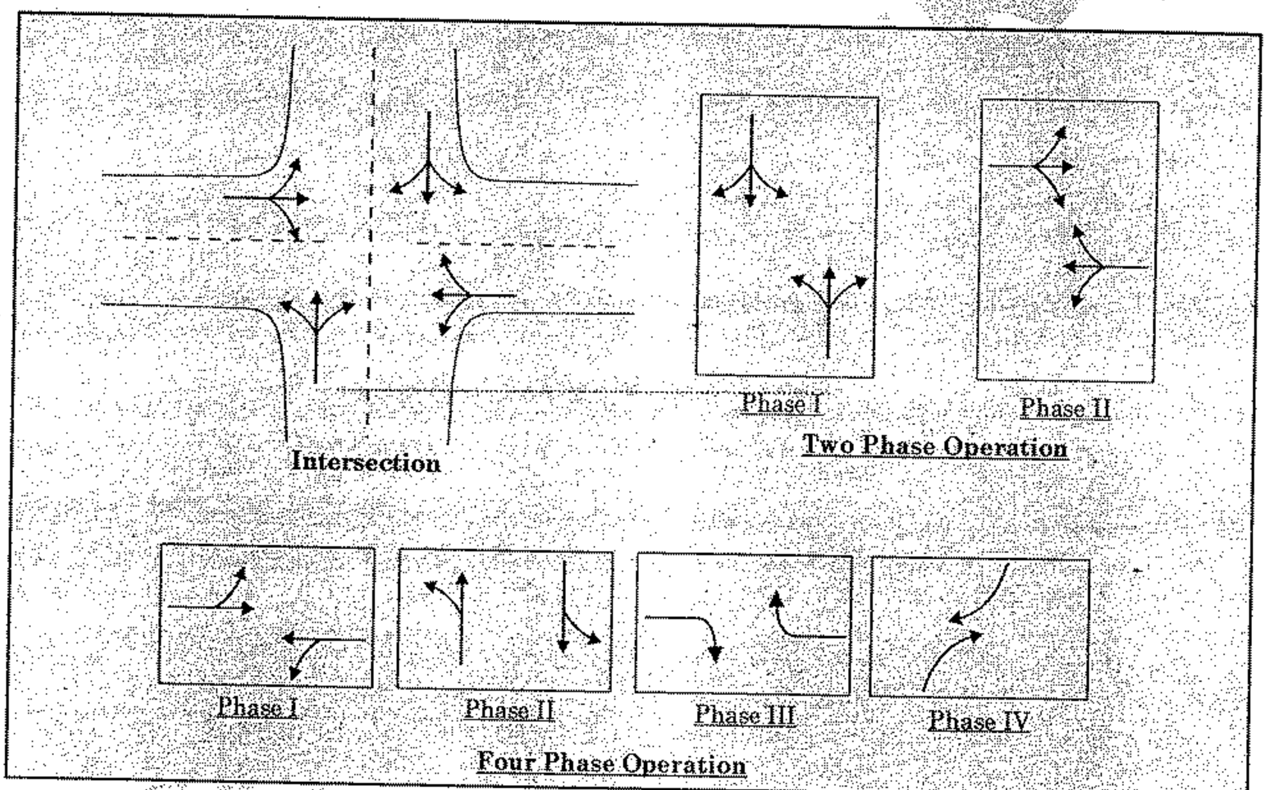
- Clearance time/interval is provided after yellow interval is used to clear off the vehicles in the intersection. Clearance interval is optional in signal design. If the intersection is small then there is no need of clearance interval :

Green Interval (G_i): The is the actual duration the green light of a traffic is turned on.

Red Interval (R_i) : The is the actual duration the red light of a traffic signal is turned on.

Phase : The sum of the displayed green, yellow, red times for a movement or combination of movements that receive the right of way simultaneously during the cycle. The sum of phase lengths (in seconds) is the cycle length.

For example in the traffic movement at intersection as shown below, the phases are shown. There can be two phase signal, 3-phase, 4-Phase, 5-Phase, 6-Phase etc.

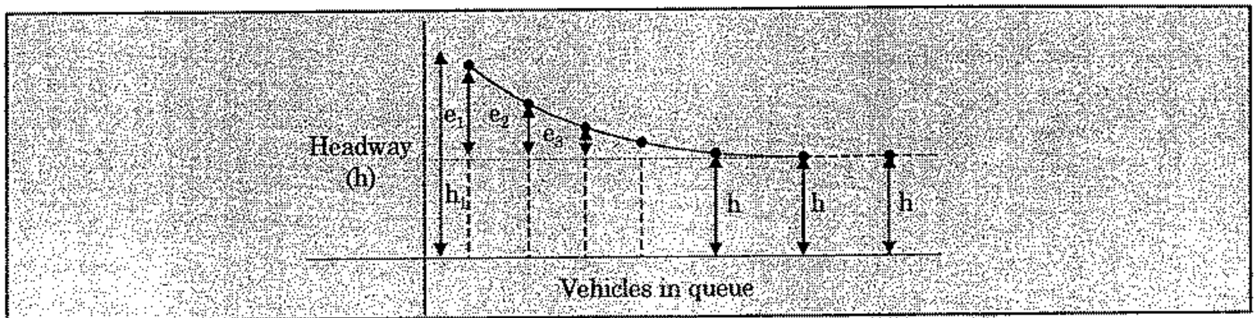


- Two Phase system is usually adopted if through traffic is significant compared to turning movements. If turning movements are significant, then a four phase system is usually adopted.
- There are various phasing options in 4-phase system. i.e. there are various ways through which the the vehicle movement is allowed in different phases.
- Five phase, six phase signals etc are normally provided if the intersection control is adaptive, that is, the signal phases and timing adapt to the real time traffic conditions.

Lost time (t_L)

- Due to traffic signal, traffic streams are continuously started & stopped. Eveytime this happens, a portion of cycle length is not being utilised completely, which translates to lost time (i.e. time that is not effectively serving any movement of traffic).
- Total lost time is a combination of
  (a) start-up lost time (b) clearance lost time

- **Startup lost time** occurs because when a signal indication turn from red to green, drivers in the queue do not instantly start moving ; there is an initial lag due to reaction to the change of signal indication.
- This startup delay result in a portion of the green time for that movement not being utilised.



- The figure above shows a group of N-vehicle at a signalised intersection waiting for green signal.
- As the signal is initiated, the time interval between two vehicles, refered as headway is noted.
- The 1st headway is the time interval between the initiation of the green signal and the instant vehicle crossing the curb line.
- Second headway is the time interval between the 1st and the second vehicle crossing the curb line.
- Successive headways are then plotted. Ist headway will be relatively longer since it includes the reaction time of the driver and the time necessary to accelerate.
- 2nd headway will be comparatively lower, because the second driver can overlap his reaction time with that of the 1st driver.
- After few vehicles, the headway will become constant.
- This constant headway is called saturation headway (h) in second

$$\boxed{\text{Saturation flow rate(Si)} = \frac{3600}{h}\frac{\text{veh}}{\text{hr}}} \qquad \boxed{\text{Start - up lost time } (t_{sl}) = \sum_{i=1}^{n} e_i}$$

- When the signal turn from green to yellow, the later portion of time during yellow interval is generally not utilised by traffic.
- Additionally, if there is an All-red interval, this time period is generally not utilised by traffic.
- These period of time during the change & clearance interval that are not effectively utilised by traffic are referred to as clearance lost time.
- However for intersection with significant red-light running, the clearnace lost time may be negligible.
- Thus $t_L = t_{sl} + t_{cL}$

  t_L = Total lost time during a cycle in second.

  t_{sl} = Startup lost time in seconds.

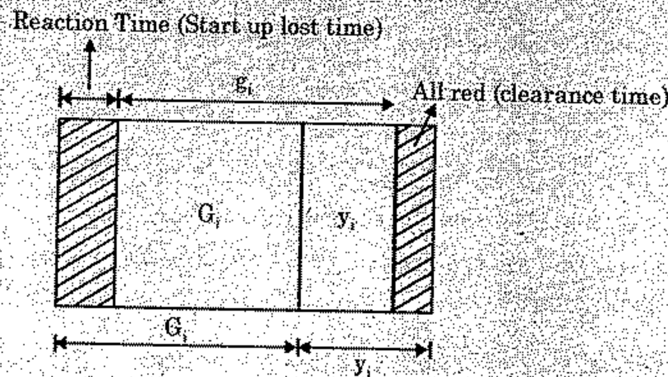
  t_{cl} = clearance lost time in seconds.

EFFECTIVE GREEN TIME

- Effective green time is the actual time available for vehicle to cross the intersection.
- It is the sum of actual green time plus yellow time minus time lost.

$$g_i = G_i + y_i - t_L$$

g_i = Effective green time
G_i = Actual Green Time
y_i = Yellow time
t_L = Lost time



CAPACITY OF A LANE

- If C = cycle length (in second), then no. of cycles in an hour is $\frac{3600}{C}$.
- Total effective green time in an hour = $\frac{3600}{C} \times g_i$.
- If h = time headway is seconds for the vehicles crossing the intersection, then total no. of vehicles crossing the intersection in one hour = $\frac{\frac{3600}{C} g_i}{h} = \frac{3600}{h} \frac{g_i}{C}$

$$\boxed{\text{capacity of a lane in } \frac{\text{veh}}{\text{hr}} = \frac{3600}{h} \frac{g_i}{C}}$$

$\frac{3600}{h}$ = Saturation Capacity , $\frac{g_i}{C}$ = green ratio.

Example 20

Cycle time of an intersection is 60 sec., Green time is 27 sec, corresponding yellow time is 4 sec. If the saturation headway is 2.4 sec. per vehicle, the start-up loss time is 2 sec. and clearance time is 1 sec., find the capacity of movement per lane.

Sol. C = 60 sec ; G_i = 27 sec. ; yi = 4 sec

$$h = 2.4 \frac{\text{sec}}{\text{veh.}}$$

Startup loss time = 2 sec.

g_i = effective green time

$$g_i = G_i + y_i - t_L$$
$$= 27 + 4 - (2 + 1) = 28 \text{ sec.}$$

$$\text{Green ratio} = \frac{28}{60}$$

$$\text{Actual capacity per lane} = \frac{3600}{2.4} \times \frac{28}{60} = 700 \text{ veh./hr/lane.}$$

TIME REQUIRED TO CLEAR "N" NUMBER OF VEHICLE

Time required to clear "N" of vehicles is equal to T, where

$$\boxed{T = N \times h + l}$$

N = Number of vehicles

h = Saturation headway (Time headway)

l = startup loss time.

CRITICAL LANE VOLUME

- During any green signal phase, several lanes on one or more approaches are permitted to move. One of these will have more intense traffic. Thus it requires more time than any other lane moving at the same time. If sufficient time is allocated for this lane, than all other lanes will also be well accomodated. There will be one and only one critical lane in each signal phase. The volume of this critical lane is called critical lane volume.

DETERMINATION OF CYCLE LENGTH

- If "t_{Li}" is the start-up loss time for phase "i", then for "n" number of phases.

Total startup loss time for all phases = $\sum_{i=1}^{n} t_{Li}$

- If cycle length is equal to C, then Number of cycles per hour = $\frac{3600}{C}$

$\Rightarrow$ Total time lost in hour = $\frac{3600}{C} \times \sum_{i=1}^{n} t_{Li}$

- However, if all phases have equal start-up loss time then

$$\sum_{i=1}^{n} t_{Li} = n \times t_L$$

n = Number of phase

t_L = Start-up loss time for each phase.

- The remaining time for movement of vehicle in one hour. (T_g) is given by

$$T_g (\text{sec}) = 3600 - \frac{3600 \times n \times t_L}{C}$$

$$\Rightarrow \quad T_g (\text{sec}) = 3600\left[1 - \frac{nt_L}{C}\right]$$

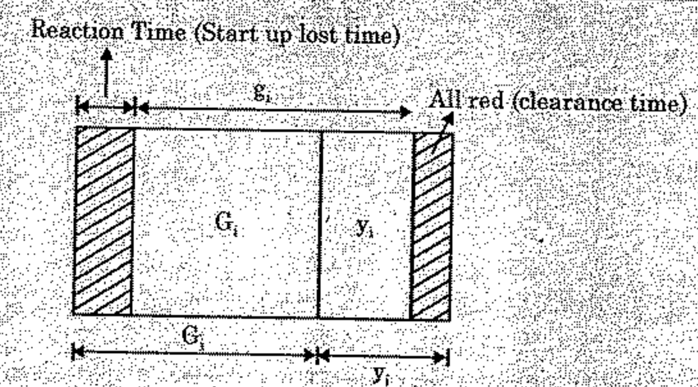
Tg = remaining time for movement (in seconds)

EFFECTIVE GREEN TIME

- Effective green time is the actual time available for vehicle to cross the intersection.
- It is the sum of actual green time plus yellow time minus time lost.

$$g_i = G_i + y_i - t_L$$

g_i = Effective green time
G_i = Actual Green Time
y_i = Yellow time
t_L = Lost time



CAPACITY OF A LANE

- If C = cycle length (in second), then no. of cycles in an hour is $\frac{3600}{C}$.
- Total effective green time in an hour = $\frac{3600}{C} \times g_i$.
- If h = time headway is seconds for the vehicles crossing the intersection, then total no. of vehicles crossing the intersection in one hour = $\frac{\frac{3600}{C} g_i}{h} = \frac{3600}{h}\frac{g_i}{C}$.

$$\boxed{\text{capacity of a lane in } \frac{\text{veh}}{\text{hr}} = \frac{3600}{h}\frac{g_i}{C}}$$

$\frac{3600}{h}$ = Saturation Capacity, $\frac{g_i}{C}$ = green ratio.

Example 20

Cycle time of an intersection is 60 sec., Green time is 27 sec, corresponding yellow time is 4 sec. If the saturation headway is 2.4 sec. per vehicle, the start-up loss time is 2 sec. and clearance time is 1 sec., find the capacity of movement per lane.

Sol. C = 60 sec., G_i = 27 sec., yi = 4 sec

$$h = 2.4 \frac{\text{sec}}{\text{veh.}}$$

Startup loss time = 2 sec.

g_i = effective green time

$$g_i = G_i + y_i - t_L$$
$$= 27 + 4 - (2 + 1) = 28 \text{ sec.}$$

$$\text{Green ratio} = \frac{28}{60}$$

$$\text{Actual capacity per lane} = \frac{3600}{2.4} \times \frac{28}{60} = 700 \text{ veh./hr/lane.}$$

TIME REQUIRED TO CLEAR "N" NUMBER OF VEHICLE

Time required to clear "N" of vehicles is equal to T, where

$$\boxed{T = N \times h + l}$$

N = Number of vehicles

h = Saturation headway (Time headway)

l = startup loss time.

CRITICAL LANE VOLUME

- During any green signal phase, several lanes on one or more approaches are permitted to move. One of these will have more intense traffic. Thus it requires more time than any other lane moving at the same time. If sufficient time is allocated for this lane, than all other lanes will also be well accomodated. There will be one and only one critical lane in each signal phase. The volume of this critical lane is called critical lane volume.

DETERMINATION OF CYCLE LENGTH

- If "t_{Li}" is the start-up loss time for phase "i", then for "n" number of phases.

Total startup loss time for all phases = $\sum_{i=1}^{n} t_{Li}$

- If cycle length is equal to C, then Number of cycles per hour = $\frac{3600}{C}$

$\Rightarrow$ Total time lost in hour = $\frac{3600}{C} \times \sum_{i=1}^{n} t_{Li}$

- However, if all phases have equal start-up loss time then

$$\sum_{i=1}^{n} t_{Li} = n \times t_L$$

n = Number of phase

t_L = Start-up loss time for each phase.

- The remaining time for movement of vehicle in one hour. (T_g) is given by

$$T_g\,(\text{sec}) = 3600 - \frac{3600 \times n \times t_L}{C}$$

$$\Rightarrow \quad T_g(\text{sec}) = 3600\left[1 - \frac{nt_L}{C}\right]$$

Tg = remaining time for movement (in seconds)

- Max. sum of critical lane volume would be equal to $V_C = \frac{Tg}{h}$, where h = saturation headway in sec.

$$V_C \text{ (veh/hr)} = \frac{Tg}{h} = \frac{3600}{h}\left[1-\frac{nt_L}{C}\right]$$

- $\frac{Tg}{h}$ is called maximum sum of critical lane volume that can be accomodated per hour.

$$\frac{3600}{h} = \text{saturation volume} = S \text{ (veh/hr)}$$

$$S\left(1-\frac{nt_L}{C}\right) = V_C$$

$$1-\frac{nt_L}{C} = \frac{V_C}{S}$$

$$1-\frac{V_C}{S} = \frac{nt_L}{C}$$

$$\boxed{C = \frac{nt_L}{1-\frac{V_C}{S}}}$$

C = cycle length (sec)

t_L = start-up loss time for each phase (sec)

n = number of phases

V_C = max. sum of critical lane volume (veh/hr) that can be accomodated per hour.

S = Saturation volume (veh/hr).

- If saturation flow for different phase is different, then

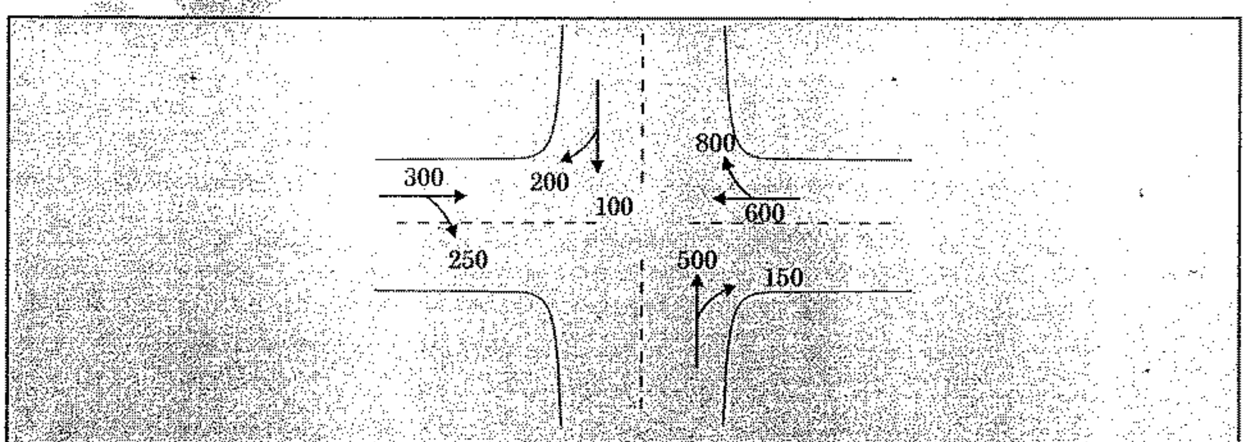
$$\boxed{C = \frac{nt_L}{1-\sum_{i=1}^{n}\frac{V_i}{S_i}}} \quad \text{..... (B)}$$

where n = no. of phases

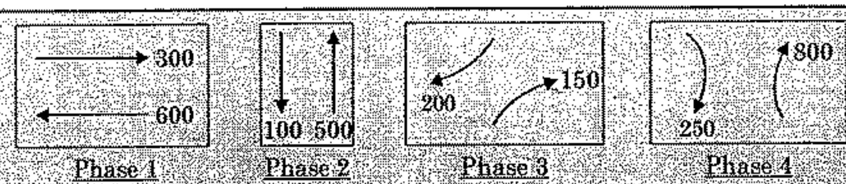
V_i = critical lane volume for i^{th} phase

S_i = saturation flow for i^{th} phase

- For example, if the traffic flow at inersection is as shown below :



and if there are 4-Phases like



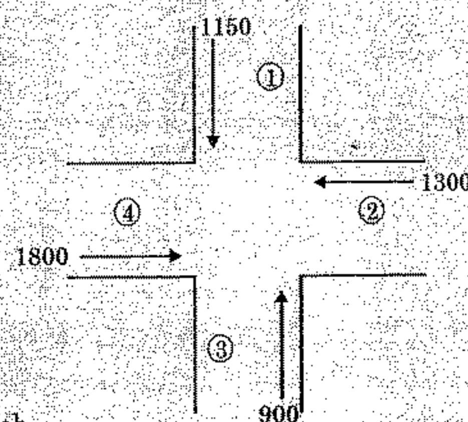
then $$\Sigma\frac{V_i}{S_i} = \frac{600}{S_1}+\frac{500}{S_2}+\frac{200}{S_3}+\frac{800}{S_4}$$

- Further, if denominator in eq. (B) becomes negative, the traffic is to be split by providing extra lane. Thus, critical lane volume change and hence denominator of eq. (B) becomes (+)ve.

The following example will illustrate this concept.

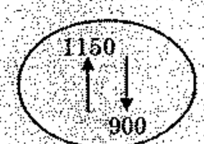
Example 22

Startup lost time = 3 sec
Saturation headway = 2.3 sec
Compute cycle length. Assume two phase signal



Sol.: Saturation headway (S) $= \frac{3600}{2.3} = 1566\frac{\text{Veh}}{\text{hr.}}$

1st Phase



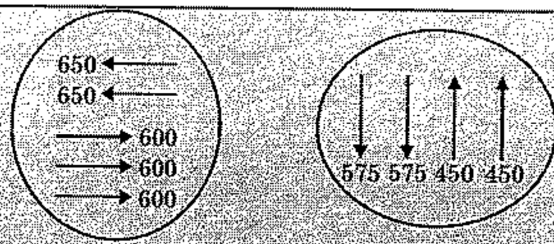
2nd Phase

Critical lane volume for 1st phase = 1800 and critical lane volume for 2nd phase = 1150.

$$\sum\frac{Vi}{Si} = \frac{1800}{1566}+\frac{1150}{1556}>1$$

⇒ Traffic needs to be split.

- We have to split the traffic in such a way that $\sum\frac{Vi}{Si}$ becomes less than 1
- If we split the traffic in the following manner i.e. Road (1) & (2) has two lanes for both the traffic in two directions and Road (2) & (4) has 3-lanes on one side and two on other side then the critical lane volume for the two phases will become 650 & 595.



$$\sum \frac{Vi}{Si} = \frac{650+575}{1566} = 0.782$$

$$C = \frac{2 \times 3}{1-0.782} = 27.5 \text{sec}$$

- Expression for cycle length in equation (B) has been obtained under the assumption that there will be uniform flow of traffic in an hour. There will be uniform flow of traffic in an hour. To account for the variation in volume in an hour, a factor called peak hour factor (PHF) which is the ratio of hourly volume to the maximum flow rate is introduced.
- Another factor $\frac{V}{C}$ ratio indicating the quality of service is also included in the equation. Incorporating these two factors.

$$\boxed{C = \frac{N t_L}{1 - \frac{V_c}{S_i \times PHF\,(v/c)}}}$$

- Highway capacity manual gives the following formula per cycle length.

$$\boxed{C = \frac{N \cdot L X_C}{X_C - \sum_{i=1}^{N} \frac{V_i}{S_i}}}$$

N = no. of phases

L = lost time per phase

$\frac{V_i}{S_i}$ = ratio of volume to saturation flow for ith phase

X_C = quality factor called critical V/C ratio i.e. critical volume -capacity ratio.

Note : In this case we will to keep $\sum \frac{V_i}{S_i}$ less than X_C by splitting the traffic as discined earlier in example.

GREEN SPLITTING

- Green splitting is the proportioning of effective green time in each of the signal phase. Green splitting is given by

$$g_i = \frac{Vci}{\sum Vci} \times Tg$$

$$Tg = C - \sum t_{Li}$$

$$= C - n t_L$$

$$gi = Gi + yi - t_{Li}$$

V_{C_i} = Critical lane volume for ith phase.

T_g = Total effective green time of a cycle.

g_i = effective green time for ith phase.

C = cycle length in second.

t_{L_i} = lost time of ith phase.

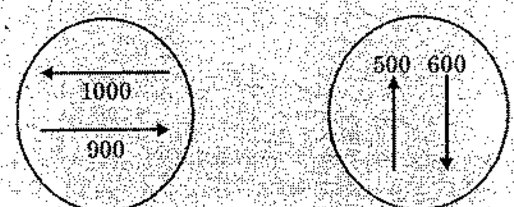
G_i = actual green time for ith phase.

y_i = yellow time or amber time for ith phase.

- From effective green time we can calculate actual green time.

Example 23

Phase diagram with flow values of an intersection with two phase is an shown below. For the lost time & yellow time given for phase I & II, find the green time allocated to each phase. Take cycle length to be 120 sec.



Phases I	Phases II
Lost time=2.5 sec	Lost time=3.5 sec
Yellow time=3sec	Yelow time=4 sec

Sol.

$$Tg = C - 2.5 - 3.5$$
$$Tg = 120 - 2.5 - 3.5$$
$$Tg = 114 \text{ sec.}$$
$$V_{C_1} = 1000$$
$$V_{C_2} = 600$$
$$\Sigma V_{C_i} = 1600$$
$$g_1 = 114 \times \frac{1000}{1600} = 71.25 \text{ sec}$$
$$g_2 = 114 \times \frac{600}{1600} = 42.75 \text{ sec}$$
$$G_1 = g_i - y_i + t_{Li}$$
$$= 71.25 - 3 + 2.5$$
$$G_1 = 70.75 \text{ sec}$$
$$G_2 = g_2 - y_2 + t_{L2}$$
$$= 42.75 - 4 + 3.5$$
$$G_2 = 42.25 \text{ sec.}$$

$G_1 = 70.75$	$y_1 = 3$	$R_1 = 46.25$	
$R_2 = 73.75$		$G_2 = 42.25$	$Y_R = 4$

PEDESTRAIN CROSSING REQUIREMENT :

- It can be taken care of by two ways.
  a) By providing an exclusive pedestrain phase (but because of this cycle time will increase)
  b) While the phase is red, pedestrain movement can be allowed in the phase.

Green time for Pedestrain :

Green time for pedestrian = walt time + clearance time.

$$\text{On road 1, } GP_1 = 7 + \frac{W_1}{1.2\,m/s}$$

$$GP_2 = 7 + \frac{W_2}{1.2\,m/s}$$

Where GP_1 & GP_2 are in seconds.
7 = walk time (min seven seconds)
W_1 = Width of road 1 in meter.
W_2 = Width of road 2 in meter.
1.2 m/s = crossing speed of pedestrian.
Thus 7 sec is the walk time & $\frac{W}{1.2\,m/s}$ is the clearance time.
Pedestrian can walk so long as the signal on that road in Red Hence.

$$\boxed{\begin{aligned} GP_1 &= R_1 \\ GP_2 &= R_2 \end{aligned}}$$

VARIOUS SIGNAL DESIGN METHOD

The various methods of signal design are :
a) Trial cycle method
b) Approximate method
c) Webster method
d) IRC method

a) Trial Cycle Method

- 15 minutes count n_A and n_B on road A and B are noted during the design peak hour flow.
- Assume a cycle time = C
  No. of vehicles passing in one cycle time.

$$\text{On Road A} = x_A = \left(\frac{n_A}{15 \times 60}\right)C$$

$$\text{On Road B} = x_B = \left(\frac{n_B}{15 \times 60}\right)C$$

- Average time required for one vehicle to cross the intersection is equal to time headway (2.5 sec assumed generally.)
- The green period G_A and G_B will be calculated as

$$G_A = x_A \times h = 2.5\, x_A$$

$$G_B = x_B \times h = 2.5\, x_B$$

- Total cycle time = $G_A + R_A + A_A$

Hence $C_1 = G_A + A_A + (G_B + A_B)$ (Note that $R_A = G_B + A_B$)

- If the calculated cycle time C_1 is equal to assumed cycle time C then the calculated cycle time is taken as the actual or design cycle time.

Example 21

A 15 min traffic count on cross road 1 and 2 during peak hours are observed as 178 and 142 vehicles per lane respectively approaching the intersection in the direction of heavier traffic flow. If the amber times required are 3 and 2 sec respectively for the two roads, design signal timing by trial cycle method. Assume average headway of 2.5 sec.

Sol.

Assume $C = 60$ sec.

$$x_1 = \frac{n_1}{15 \times 60} \times 60 = 11.87$$

$$G_1 = 11.87 \times 2.5 = 29.67$$

$$x_2 = \frac{n_2}{15 \times 60} \times 60 = \frac{142}{15} = 9.47$$

$$G_2 = 9.47 \times 2.5 = 23.67$$

$$C_1 = G_1 + A_1 + G_2 + A_2 = 29.67 + 3 + 23.67 + 2$$

$$= 58.34 \neq C \text{ (assumed)}$$

We will revise the assumption and we will perform trial and error till $C_1 = C$.

Let us assume $C = 45$ sec

$$\Rightarrow \quad x_1 = \frac{178}{15 \times 60} \times 45 = 8.9$$

$$G_2 = 8.9 \times 2.5 = 22.25 \text{ sec}$$

$$x_2 = \frac{142}{15 \times 60} \times 45 = 7.1$$

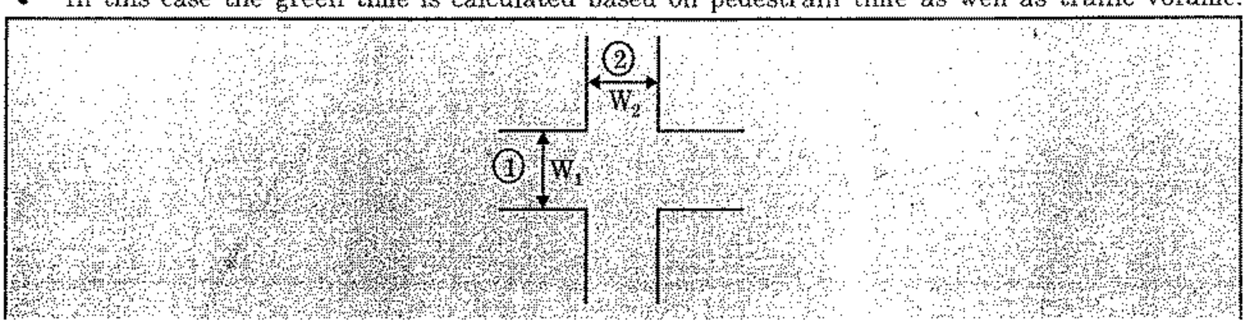
$$G_2 = 7.1 \times 2.5 = 17.75$$

$$C_1 = G_1 + A_1 + G_2 + A_2 = 22.25 + 3 + 17.75 + 2 = 45 \text{ sec}$$

Then as $C_1 = C$ (annum) the correct cycle time is 45 sec.

APPRIXIMATE METHOD FOR TWO PHASE SIGNAL ALONG WITH PEDESTRIAN SIGNAL

- In this case the green time is calculated based on pedestrain time as well as traffic volume.



Green time for pedestrian on Road 1 = $GP_1 = \frac{W_1}{1.2} + 7\,\text{sec}$

Green time for pedestrian on Road 2 $GP_2 = \frac{W_2}{1.2} + 7$

$$\boxed{\begin{matrix} GP_1 = R_1 \\ GP_2 = R_2 \end{matrix}} \text{ Minimum Red time}$$

$$G_1 + A_1 = R_2 \text{ (For two phase signal)}$$

$$\begin{matrix} \Rightarrow \boxed{G_1 = R_2 - A_1} \\ \Rightarrow \boxed{G_2 = R_1 - A_2} \end{matrix} \text{ Minimum green time}$$

- But green time should also account for vehicular movement. Hence following steps are taken for calculation of green time.
- If "n_1 and n_2" are approach volume of heaviest traffic per hour per lane on road (1) and (2) respectively then,

$$\frac{G_1}{G_2} = \frac{n_1}{n_2}$$

(i.e. green time on a road should be in proportion to the traffic volume on that road)

- If G_1 is taken from vehicular consideration, then

$$G_2' = G_1 \cdot \frac{n_2}{n_1}$$

G_2' should not be less than. G_2 from pedestrain movement consideration.

- If it is less than G_2 from Pedestrain movement consideration, take G_2 as that from Pedestrain movement and findout G_1. Thus this new value of G_1' will be

$$G_1' = G_2 \times \frac{n_1}{n_2}$$

These new values of G_1 and G_2 are then adopted

- Following example will illustrate the point discussd above

If from pedestrain movement criteria $G_1 = 20$ $G_2 = 10$ and $n_1 = 30$ $n_2 = 5$

Take $G_1 = 20$ & find G_2'

$$G_2' = \frac{20 \times 5}{30}$$

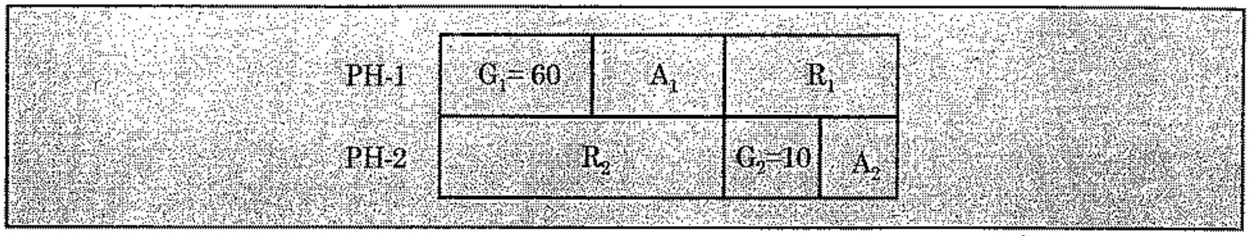
$$G_2' = 3.33 < 10 \text{ (i.e. from pedestrian movement consideration)}$$

Hence adopt $G_2 = 10$ and Find G_1'

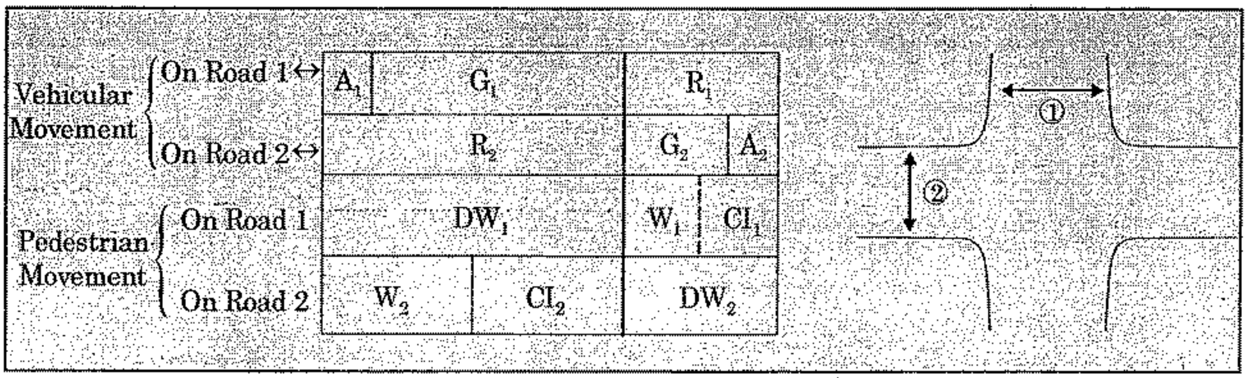
$$G_1' = G_2 \frac{n_1}{n_2} = 10 \times \frac{30}{5} = 60$$

Hence final values adopted are $G_1 = 60$

$G_2 = 10$



Pedestrain Movement :



DW = Do not walk time

W = Walk time

Cl = Clearance time

$$\boxed{DW_1 = R_2 = G_1 + A_1}$$

$$\boxed{DW_2 = R_1 = G_2 + A_2}$$ (Do not walk period for pedestrian)

$$\boxed{CI_1 = \frac{W_1}{1.2}}$$ (Clearance internal for pedestrian)

$$\boxed{CI_2 = \frac{W_2}{1.2}}$$

$$\boxed{W_1 = R_1 - CI_1}$$

$$\boxed{W_2 = R_2 - Cl_2}$$ (Walk Period for pedestrial)

W_1 = Width of road 1 in meter

1.2 = speed of pedestrian in m/sec

Note :

- If Pedestrain signal is used, walk period should not be less than 7 seconds.
- With no Pedestrain signal, minimum walk period is 5 seconds.
- Cycle time should be in multiple of 5 seconds.
- Any deviation should be distributed in green time in proportion to the traffic volume.

Example 24

For the data given below, design the timings of traffic and pedestrian signals using approximate method.

	Road A	Road B
width	21 m	15 m
Highest volume from one direction	350 Veh / hr / Lane	260 Veh. / hr / Lane
Amber Time 50 sec	5 sec	5 sec

Design with approx method :

$$GP_A = 7 + \frac{21}{1.2} = 24.5 \text{ sec.}$$

$$GP_B = 7 + \frac{15}{1.2} = 19.5 \text{ sec.}$$

$$R_A = GP_A = 24.5 \text{ sec.}$$

$$R_B = GP_B = 19.5 \text{ sec.}$$

$$G_A = R_B - A_A = 19.5 - 5 = 14.5 \text{ sec.}$$

$$G_B = R_A - A_B = 24.5 - 5 = 19.5 \text{ sec.}$$

$$\frac{n_A}{n_B} = \frac{350}{260} = 1.35$$

Take

$$G_A = 14.5$$

$$G_B' = \frac{n_B}{n_A} G_A = \frac{14.5}{1.35} = 10.75 < 19.5$$

Adopt $G_B = 19.5$

$$G_A' = \frac{n_A}{n_B} \times G_B$$

$$= 1.35 \times 19.5 = 26.325 > 14.5 \text{ sec}$$

Hence adopt

$$G_A = 26.325$$

$$G_B = 19.50$$

$$A_A = 5 \text{ sec.}$$

$$A_B = 5 \text{ sec.}$$

$$\text{Total Cycle Time} = 55.825$$

Adopt cycle time as 60 sec.

Deviation = 60 − 55.825 = 4.175 sec.

- It will be distributed in G_A and G_B in proportion to traffic volume.

$$\Delta G_A = \frac{n_A}{n_A + n_B} \times 4.175 = 2.395 \text{ sec}$$

$$\Delta G_B = 4.175 - 2.395 = 1.78 \text{ sec}$$

$$\Rightarrow \quad G_A = 26.325 + 2.395 = 28.72 = 29 \text{ sec.}$$

$$G_B = 19.5 + 1.78 = 21.28 \text{ sec.} = 21 \text{ sec}$$

$$\text{Total Cycle Time} = C = G_1 + A_1 + A_2 + G_2 = 60 \text{ sec.}$$

PH-A	$G_A = 29$	$A_A = 5$	$R_A = 26$	
PH-B	$R_B = 34$		$G_B = 21$	$A_B = 5$

60 sec.

PH-A	$DW_A = 34$		$W_A = 8.5$	$CI_A = 17.5$
PH-B	$W_B = 21.5$	$CI_B = 12.5$	$DW_B = 26$	

$$W_A = R_A - CI_A = 26 - 17.5 = 8.5 \text{ sec.}$$

$$W_B = R_B - CI_B = 34 - 12.5 = 21.5 \text{ sec.}$$

- Clearance interval would be fixed but walk period would be variable. Hence CI will be calculated first and then walk period calculated. (Note that walk period in both the pedestrian signal in greater than 7 sec hence)

WEBSTER METHOD

- In this method, optimum cycle time is calculated from least total delay at signalized intersection.

$$\text{Optimum Cycle time (sec.)} = \frac{1.5L+5}{1-Y} = C_0$$

$L = nt_L + R$

n = no. of phase

t_L = Start-up loss time and loss time due to falling of discharge rate during amber period.

R = All red time

For the average signal cycle, the lost time (t_L) amounts to around 2 sec.

$$\Rightarrow \quad L = 2n + R$$

$$Y = y_1 + y_2 + \ldots\ldots\ldots\ldots$$

$$y_1 = \frac{q_1}{S_1}$$

$$y_2 = \frac{q_2}{S_2} \ \ldots\ldots\ldots \text{ so on}$$

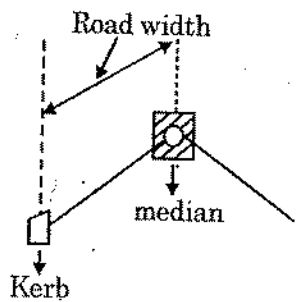
q_1 = Critical lane volume for phase one (maximum volume per lane)

S_1 = Saturation flow for phase one.

Note :

$$S_1\left(\frac{veh}{hr}\right) = \frac{3600}{\text{time headway(sec.)}}$$

- Y should not be greater than one.
- Webster method is a most rational method of design.
- The saturation flow can be assumed as given in the table below

Road Width	3 m	3.5 m	4 m	4.5 m	5 m	5.5 m	> 5.5 m
							
Saturation flow for full carriageway or road width (PCU/hr)	1850	1890	1950	2250	2350	2990	525 per m width

$$\boxed{\text{Green Time} = \frac{(C_0 - L)y_1}{y_1 + y_2 + \ldots} = G_1}$$

$$\boxed{= \frac{(C_0 - L)y_2}{y_1 + y_2 + \ldots} = G_2}$$

Example 25

$$q_A = 400 \text{ PCU/hr.}$$

$$q_B = 250 \text{ PCU/hr.}$$

$$\text{Saturation flow } S_A = 1250 \text{ PCU/hr.}$$

$$S_B = 1000 \text{ PCU/hr.}$$

All red time required for pedestrian crossing is 12 sec.

Design two phase signal by webster method.

Sol.

$$y_A = \frac{q_A}{S_A} = \frac{400}{1250} = 0.32$$

$$y_B = \frac{q_B}{S_B} = \frac{250}{1000} = 0.25$$

$$Y = y_A + y_B = 0.57$$

$$L = 2n + R$$

$$L = 2 \times 2 + 12$$

$$L = 16 \text{ sec.}$$

$$C_0 = \frac{1.5L+5}{1-Y} = \frac{1.5\times16+5}{1-0.57} = 67.44 \text{ sec.}$$

$$\text{Green time } (G_A) = \frac{(C_0-L)y_A}{y_A+y_B} = \frac{(67.44-16)0.32}{0.57} = 28.89 \text{ sec.}$$

$$G_B = \frac{(Co-L)y_B}{y_A+y_B} = 22.56 \text{ sec.}$$

$$\text{Adopt } G_A = 29$$

$$G_B = 23$$

Providing amber time of 2 sec for each phase for clearance,

$$C_0 = 68 \text{ sec}$$

$$G_A + A_A + G_B + A_B = 68 \text{ sec}$$

$$A_A + A_B = 68 - 29 - 23 = 16 \text{ sec}$$

In this A_A & A_B, the all red time (is also included).

Phase A	$G_A = 29$	2	12	Red = 25	
Phase B	$R_B = 31$		12	$G_B = 23$	AB = 2

Note : Smaller amber time may lead to dilamma. Hence it can be increased.

IRC METHOD

- In IRC method signal timining is decided by approximate method and design is checked.
- The pedestrian green time req. for major & minor roads are calculated based on working speed of 1.2 m/s and initial walk period of 7.0 sec. These are the minimum green time required for the vehicular traffic on the minor & major roads respectively.
- The green time required for the vehicular traffic on the major road is increased in proportion to the traffic on the two approaches.
- The approach followed in just as in the approximate method already discussed.
- Total cycle length is calculated as

$$C = G_A + A_A + G_B + A_B$$

$$A_A = (\text{clearance amber + initial amber})_A$$

$$A_B = (\text{clearance amber + initial amber})_B$$

Each of clearance amber & initial amber is taken as 2 sec.

Immediate higher multiple of 5 seconds is taken as the cycle time and additional seconds may be apporitioned in green time in proportion to the critical lane volume on the two roads.

Check for design signal cycle timing in the bases of vehicular volume.

- Minimum green time required for clearing the vehicle arriving during a cycle is determined for each lane of approach road assuming that first vehicle will take 6 seconds and subsequent will be clear at the rate of 2 seconds.
- If the critical volume per lane on road one is n_1 per hour then number of vehicle accumulated in one cycle:

$$x_1 = \frac{n_1}{3600} \times C$$

C = The cycle time obtained from approximate method as done before.

- Hence time required to clear x_1 vehicle (t_1) is

$$t_1 = 6 + (x_1 - 1) \times 2$$

- This time is the minimum green time required for road 1. Similarly check for other road will be done.

Note : The minimum green time for vehicular traffic is taken as 16 sec.

Check for Optimum Cycle Time

- Check for optimum cycle time is done using webster method.

$$C_0 = \frac{1.51+5}{1-y}$$

$$y = \sum y_i$$

$$y_i = \frac{q_i}{S_i}$$

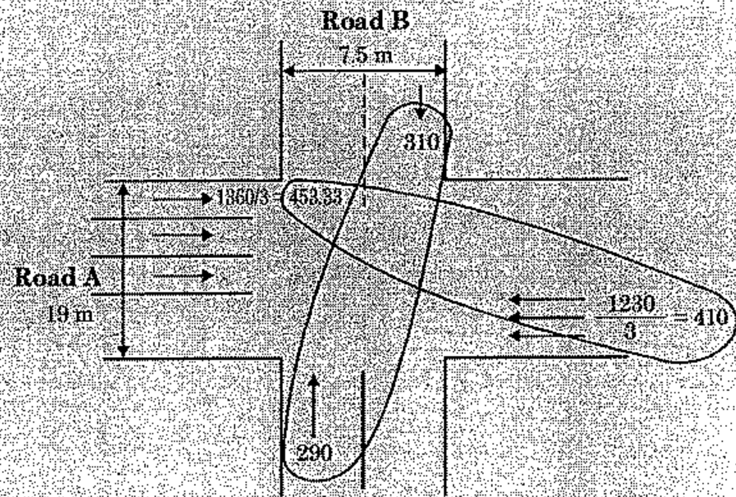
- The loss time in this case is taken as L = 2 × (4 sec.) + Total amber time in two phases.
- This cycle time should be sufficient for both vehicle as well as pedestrain. If both conditions are satisfied this optimized cycle time is adopted. However, if modification are required, the required modification is done.

Example 26

Design signal timing as per IRC method for the data given below :

	Road A	Road B
Road Width	19 m	7.5 m
No.of lane	Six	Two
Traffic in one direction (Total)	1360 Veh / hr	310 Veh / hr
Traffic in opposite direction (Total)	1230 Veh / hr	290 Veh / hr
Amber time	6	6

Sol.



Critical lane volume for phase 1 = 453.33, (critical lane volume for phase 2 = 310)

$$G_{PA} = 7+\frac{W_A}{1.2} = 7+\frac{19}{1.2} = 22.83\,\text{sec.}$$

$$G_{PB} = 7+\frac{7.5}{1.2} = 13.25\,\text{sec.}$$

$$\Rightarrow \quad R_A = 22.83\text{ sec.}$$

$$\Rightarrow \quad R_B = 13.25\text{ sec.}$$

$$G_A = R_B - A_A = 13.25 - 6 = 7.25\,\text{sec}$$

$$G_B = R_A - A_B = 22.83 - 6 = 16.83\,\text{sec.}$$

$$\frac{G_A}{G_B} = \frac{q_A}{q_B} = \frac{453.33}{310}$$

$$\text{Take } G_B = 16.83$$

$$G_A = \frac{453.33}{310}\times 16.83$$

$$G_A = 24.61\text{ sec.} > 7.2\text{ sec.}$$

Hence adopt $G_A = 24.61$ sec, $G_B = 16.83$ sec

$$\text{Cycle time} = G_A + A_A + G_B + A_B$$

$$C = 24.61 + 6 + 16.83 + 6$$

$$C = 53.44\text{ sec.}$$

The next higher multiple of 5 is 55 (difference has to be apportioned in the two green time.

$$\Delta C = 55 - 53.44 = 1.56$$

$$\text{Hence Adopt } C = 55\text{ sec.}$$

$$G_A = 24.61+\frac{1.56\times 453.33}{453.33+310} = 25.53\,\text{sec, say 26 sec}$$

$$G_B = 55 - 12 - 25.53 = 17.46\text{ sec, say 17 sec}$$

Check for design on the basis of vehicular consideration :

G_A = 26 sec.

G_B = 17 sec.

A_A = 6 sec.

A_B = 6 sec.

No. of vehicles on road "A" accumulated in cycle time of 55 sec is

$$= \frac{453.33}{3600} \times 55 = 6.92 \quad \text{say 7 nos.}$$

No. of veicles on road "B" accumulated in cycle time of 55 sec is

$$= \frac{310}{3600} \times 55 = 4.74 \text{ say 5 nos.}$$

Time required to clear 7 vehicles on road "A"

$$= 6 + (7-1) \times 2 = 18\text{sec.} < 26 \text{ sec}, \text{ OK.}$$

Time required to clear 5 vehicles on road "B"

$$= 6 + (5-1) \times 2 = 14\text{sec.} < 17 \text{ sec}, \text{ OK.}$$

Optimization of Cycle Time :

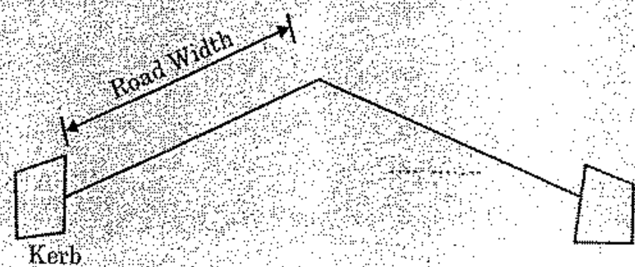
- To optimize the cycle time we will use webster method.

$$C_0 = \frac{1.5L+5}{1-Y}$$

$$L = 2 \times 6 + 2 \times 4 = 20 \text{ sec.}$$

$$Y = \frac{q_A}{S_A} + \frac{q_B}{S_B}$$

Saturation flow is obtained on the basis on road width.



$$\text{Road A, Road Width} = \frac{19}{2} = 9.5\,\text{m}$$

$$\text{Road B, Road Width} = \frac{7.5}{2} = 3.75\,\text{m}$$

Saturation flow = 525 veh/hr per meter width

$$\text{Hence saturation flow per lane} = S_A = \frac{525 \times \frac{19}{2}}{3} = \frac{4987.5}{3} = 1662.5$$

3 = No. of lane.

$$S_B = \frac{1890+1950}{2} = 1920$$

$$Y = \frac{453.33}{1662.6} + \frac{310}{1920} = 0.434$$

$$C_0 = \frac{1.5\times20+5}{1-0.434} = 61.8$$

$$C_0 = 61.8$$

- We can adopt the cycle time as 65 seconds. Let us adopt 65 second as the total cycle time.

$$G_A = \frac{(C_0 - L)y_1}{y_1 + y_2} = \frac{(65-20)\times0.273}{0.434}$$

$$G_A = 28.30 \text{ sec.}$$

Adopt $G_A = 28$ sec. ($g_1$eff)

$$G_B = \frac{(C_0 - L)y_2}{y_1 + y_2} = (C_0 - L) - G_1$$

$(65 - 20) - 28 = 17$ sec. ($g_2$eff)

Red time for $A = G_B + A_B + 4 = 17 + 6 + 4 = 27$ sec

Red time for $B = G_A + A_A + 4 = 28 + 6 + 4 = 38$ sec

$\Rightarrow$ R_A form vehiculur consideration > R_A from pedestrian criteria

$\Rightarrow$ R_B from vehicular congidration > R_B from pedestrian criteria

Thus design is ok

Hence adopt $G_A = 28 + 4 = 32$ sec

$G_B = 17 + 4 = 21$ sec

$A_A = 6$ sec

$A_B = 6$ sec

G_A=32	A_A= 6	R_A=27
R_A=38	G_B=21	A_B= 6

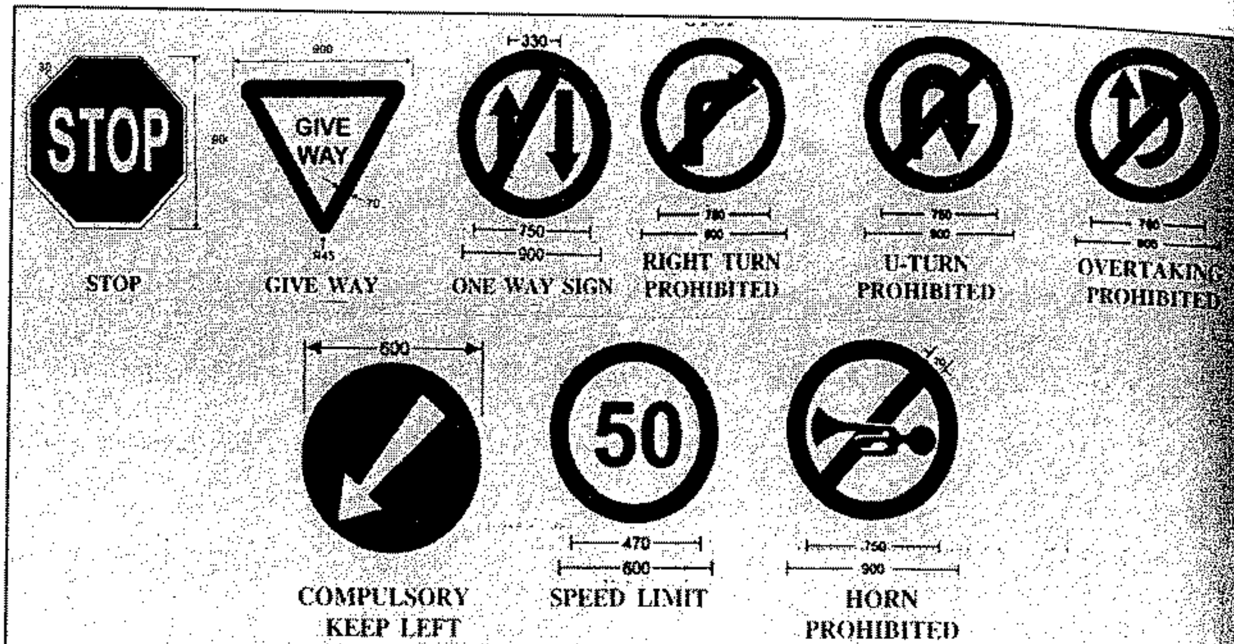
TRAFFIC SIGNS

Classification of signs

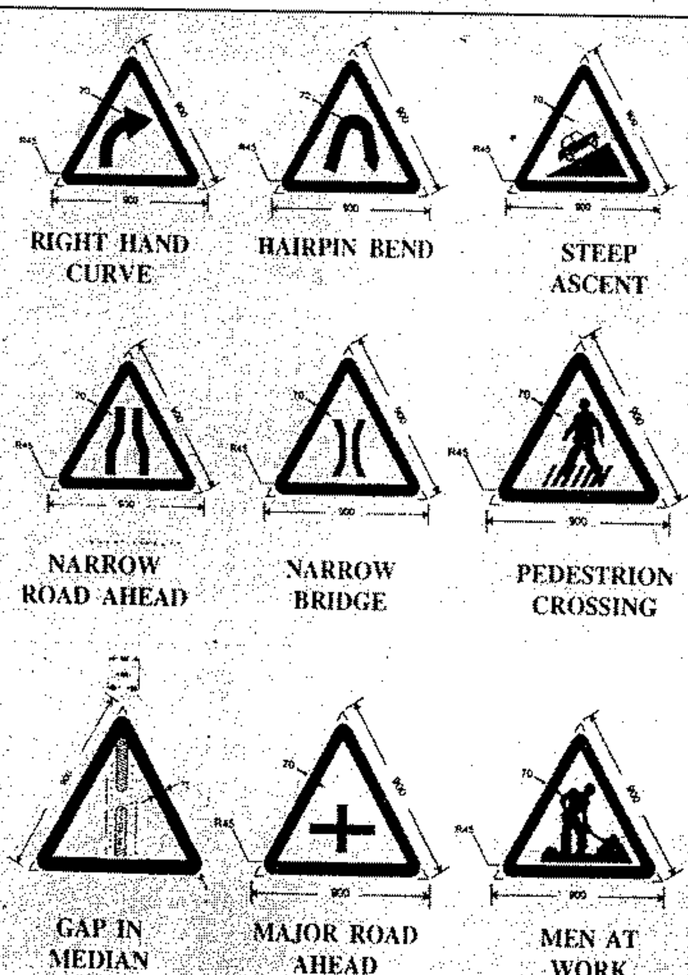
Mandatory/Regulatory : These inform the road users of laws and regulations. Violation is a legal offence.

Cautionary/Warning : Warn road users of the existence of certain hazardous conditions.

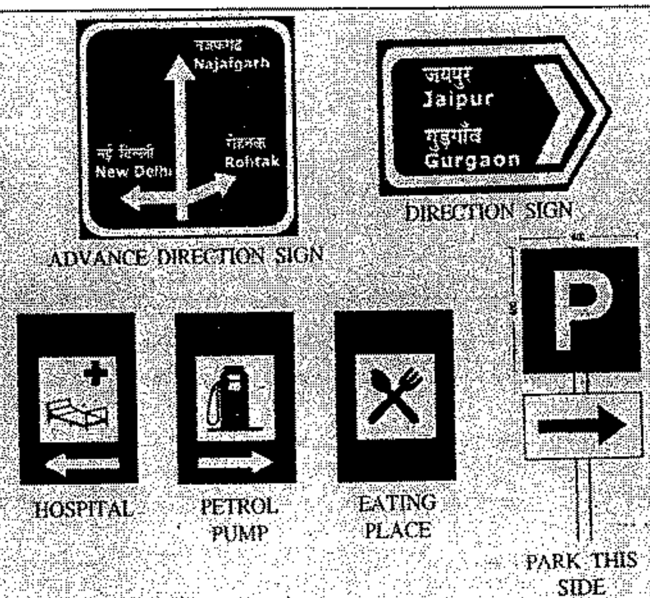
Informatory : For information and guidance of road users.



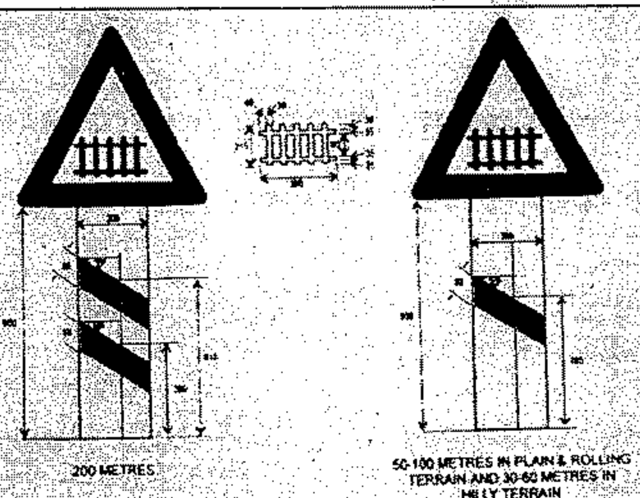
Mandatory/regulatory signs



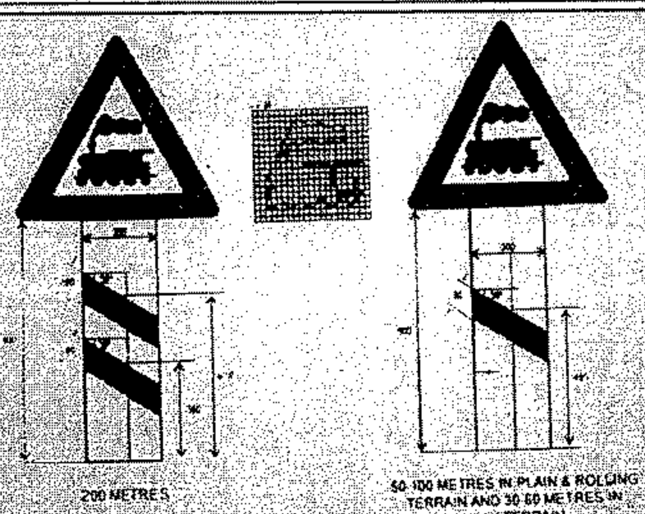
Cautionary/warning signs



Informatory signs



Guarded railway crossing



Unguarded railway crossing

ROAD MARKING

- Markings on the carriageway and on the objects within and adjacent to the roadway are used as a means of guiding and controlling the traffic.
- They promote road safety & ensure smooth flow of traffic into the required paths of travel.

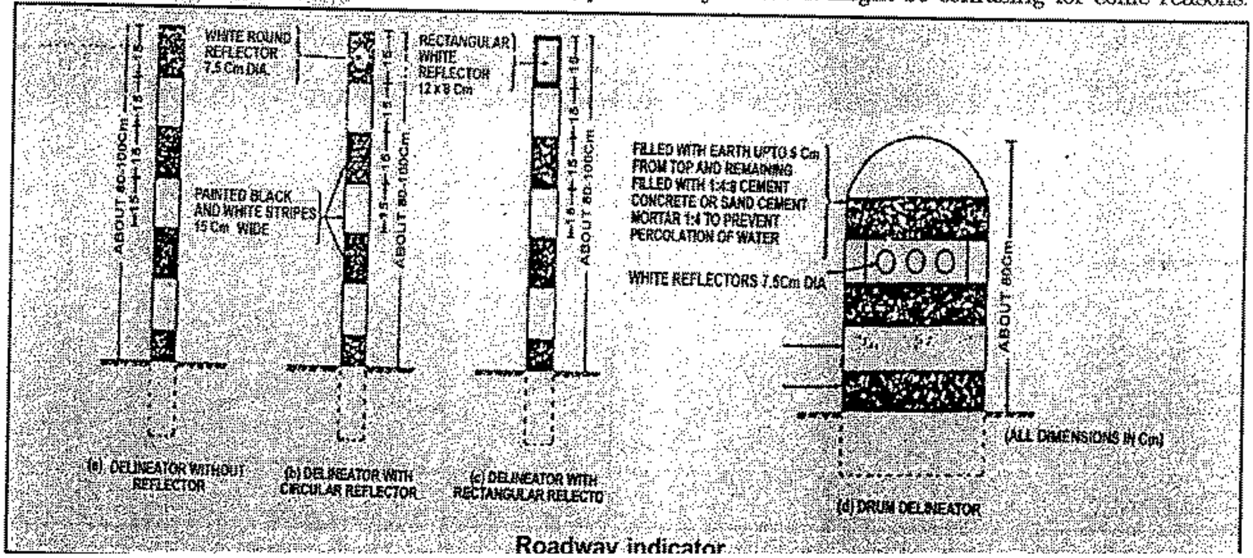
Types of carriageway marking lines

- A broken longitudinal line is used for indicating the centre line on two and three-lane roads and for lane marking on multi-lane roads. Drivers may cross these at their discretion, if traffic permits.
- Longitudinal solid lines are used as guiding or regulating lines and are not meant to be crossed by the driver except for entry or exit from a premises or a side road or to avoid a stationary obstruction.
- Double solid lines indicate maximum restriction and are not to be crossed except in emergent usage.
- In a combination of broken and solid lines, a solid line may be crossed, with discretion, if the broken line of the combination is nearer to the direction of travel. Vehicles from opposite directions are not permitted to cross the solid lines.
- Solid lines either parallel to the intersecting roadway or at right angles to the direction of approaching traffic mark the position of a stop line before the road junction. Stop line indicates the position beyond which the vehicles should not proceed when required to stop by the traffic police, traffic signals or other traffic control devices. Stop lines shall not be used unless traffic control by any one of these means exist.

ROAD DELINEATORS

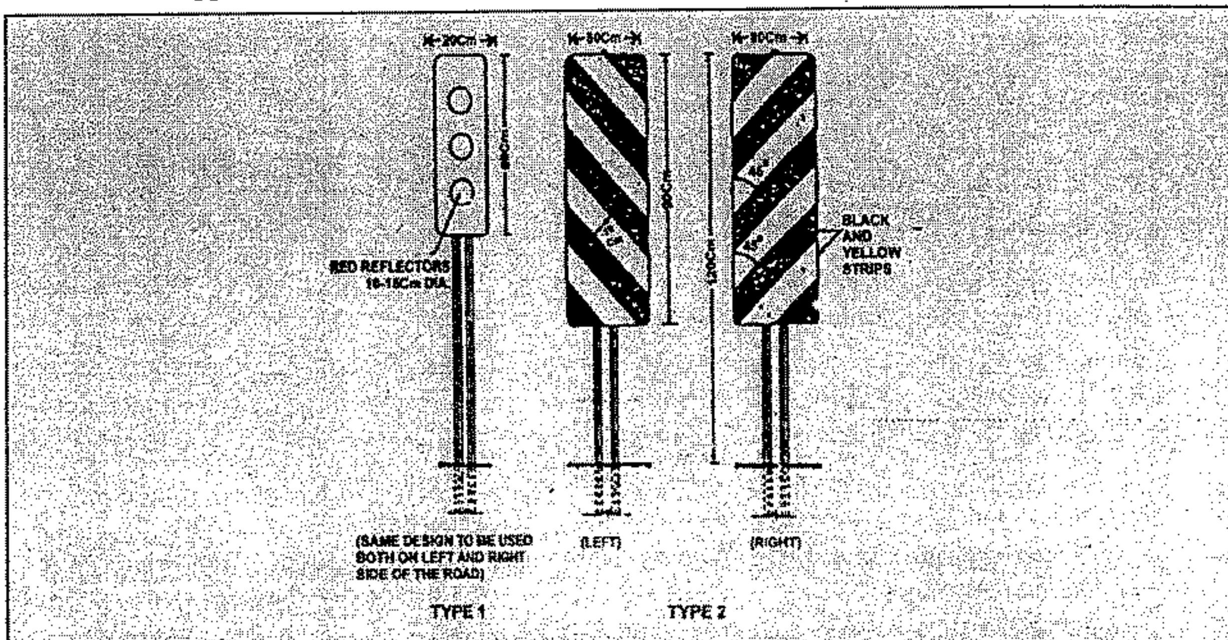
The role of delineators is to provide visual assistance to drivers about alignment of the road ahead, especially at night. Reflectors are used on the delineators for better night visibility. Delineators are classified under three types:

(i) **Roadway Indicators:** These are intended to delineate the edges of the roadway so as to guide drivers about the alignment a head, particularly where it might be confusing for some reasons.



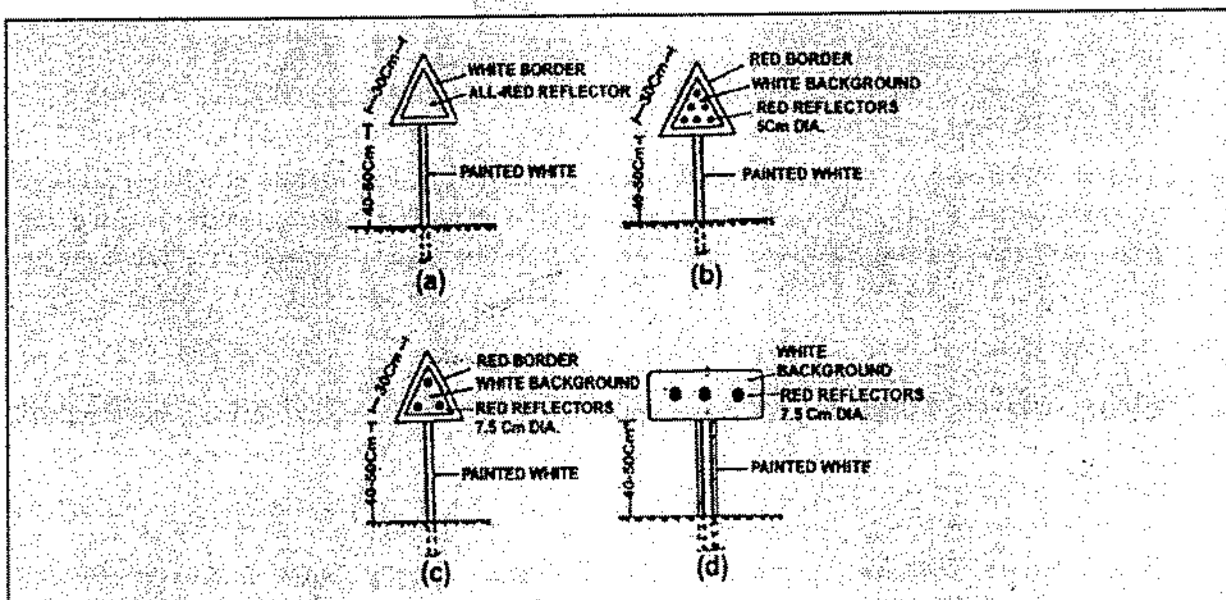
Roadway indicator

(ii) **Hazard Markers:** These are to define obstructions, like, guard-rails are and abutments adjacent to the carriageway, for instance at culverts and bridges which are narrower than the roadway width at approaches. Hazard markers are shown in Fig.



Hazards markers

(iii) **Object Markers:** These are used to indicate hazards and obstructions within the vehicle flow path, for example channelising islands close to intersections. Typicals designs of object markers are shown in Fig.



Typical designs of object markers

TRANSPORTATION MODELLING

Transportation modeling consists primarily of four steps (and is thus often referred to as the "(four-step" modeling process). These will be denoted with the following variables :

Trip Generation (T_i)	The number of trips produced in traffic analysis zone (i).
Trip Distribution (T_{ij})	The number of trips produced in zone i and attracted to zone j.
Mode Split (T_{ijm})	The number of trips produced in zone i and attracted to zone j travelling by mode m.
Trip Assignment (T_{ijmr})	The number of trips produced in zone i and attracted to zone j travellig by mode m over route r.

Trip Generation

- A trip is defined as the one way movement having single purpose and mode of travel between a point of origin & a point of destination.
- The trip generation step in transportation modeling relates the number of trips being produced from a zone or site by time period to the land sue and demographic characteristics found at the location.
- A necessary input step into trip generation is to have some indication of what the land use and demographic characteristics are likely to be.
- For future conditions, special models are used to estimate population, number of dwelling units, auto ownership, income, employment, retail sales and other factors that will likely characterize the future conditions for this zone.
- In many cases, and especially for regional or statewide planning, these future estimates are provided by economic or demographic planners rather than by transportation professionals.
- Studies have shown that the rate of trip making is closely related to three characteristics of land use : (1) intensity of land use (e.g. dwelling units per acre, employees per acre, etc.), (2) character of land use (e.g. average family income, car ownership, etc.), and (3) location relative to major economic activities (e.g. closeness to downtown).

Trip Distribution

- The major product of the trip distribution step in transportation modeling is the trip table, an origin-destination matrix that shows the number of trips originating in the study zones and where these trips are destined to. The major method of producing such trip tables is the gravity model.

Mode Split Models

Mode split models predict the number of travelers who will choose one mode over others for making a particular trip. Empirical evidence indicates that the following factors influence mode choice.

1. Type of trip (e.g. trip purpose, time of day)
2. Characteristics of the tripmaker (e.g. income, age, auto ownership)
3. Characteristics of the transportation system (e.g. relative travel times for the modes available to make the trip)

Trip Assignment

The final step in transportation modeling is to assign trips to paths in the network. The most important concept in trip assignment is that travelers will choose a path that minimizes travel time from origin to destination. Trip assignment models therefore, are based on minimum time algorithms that identify the minimum time paths through networks.

OBJECTIVE QUESTIONS

1. In desire-line diagram
   (a) width of desire-line is proportional to the number of trips in one direction
   (b) length of the desire-line is proportional to the number of trips in both directions
   (c) width of desire-line is proportional to the number of trips in both direction
   (d) both length and width of desire-line are proportional to the number of trips in both directions

2. Consider the following situations:
   1. Traffic volume entering from all roads is less than 3000 vehicles per hour.
   2. Pedestrian volume is high.
   3. Total right turning traffic is high.
   4. A road in a hilly region.

   A rotary will be more suitable than control by signals, in situations listed against
   (a) 1 and 3 (b) 1 and 4
   (c) 2 and 4 (d) 2 and 3

3. Ratio of the width of the car parking area required at kerb for 30° parking relative to 60° parking is approximately
   (a) 0.5 (b) 0.7
   (c) 0.8 (d) 2.0

4. For the relationship $u = 55 - 0.44\,k$, where 'u' is the speed in kmph and 'k' is the density in vpkm, what will be the maximum flow in vph?
   (a) 1718 (b) 1250
   (c) 625 (d) 125

5. Consider the following statements:
   Collision diagram is used to
   1. study accident pattern
   2. eliminate accidents
   3. determine remedial measures
   4. make statistical analysis of accidents

   Which of these statements are correct?
   (a) 1 and 2 (b) 1 and 3
   (c) 3 and 4 (d) 2 and 4

6. Consider the following parameters related to a rotary intersection:
   1. Width of the weaving section.
   2. Length of the weaving section.
   3. Proportion of weaving traffic.
   4. Weaving angle.
   5. Width of the carriageway at entry.

   Capacity is generally expressed in terms of
   (a) 1, 2, 3 and 4 (b) 1, 2, 3 and 5
   (c) 1, 2 and 3 (d) 4 and 5

7. It was noted that on a section of road, the free speed was 80 kmph and the jam density was 70 vpkm. The maximum flow in vph that could be expected on this road is
   (a) 800 (b) 1400
   (c) 2800 (d) 5600

8. If the normal flows on two approach roads at an intersection are respectively 500 pcu per hr and 300 pcu per hr, the saturation flows are 1600 pcu per hr on each road and the total lost time per signal cycle is 16 s, then the optimum cycle time by Webster's method is
   (a) 72.5 s (b) 58 s
   (c) 48 s (d) 19.3 s

9. When two roads with two-lane, two-way traffic, cross at an uncontrolled intersection, the total number of potential major conflict points would be
   (a) 32 (b) 24
   (c) 16 (d) 4

10. **Assertion (A):** IRC suggest that the maximum volume of traffic of 3000 vehicles per hour entering from all legs of the rotary intersection can be handled efficiently.
    Reason (R): Traffic rotaries may be provided where the intersecting traffic is about 50% of the total traffic or fast turning traffic is at least 30% of the total traffic.

11. Consider the following factors:
    1. Length of vehicle
    2. Width of vehicle
    3. Approach speed
    4. Stopping time for approaching vehicle
    5. Passing sight distance

    Which of these factors are taken into consideration for determining yellow time of a traffic signal at an inter section?
    (a) 1, 2 and 5 (b) 2, 3 and 4
    (c) 1, 3 and 5 (d) 1, 3 and 4

12. An observer travelling at a constant speed of 70 kmph with the traffic stream over a 5 km stretch is passed by 17 vehicles more than what he passes. When the observer travels against the stream at the same speed, the number of vehicles he meets is 303. The flow of the traffic stream is
    (a) 4480 vph (b) 4160 vph
    (c) 2240 vph (d) 2002 vph

13. If the total seven-day traffic volume for the week is 3625 and the traffic volume for Monday is 650, then the monthly expansion factor for Monday would be
    (a) 5.6 (b) 1.25
    (c) 0.8 (d) 0.25

14. Which one of the following is the purpose of divisional island?
    (a) To divert the traffic into definite travel path at the intersection.
    (b) To reduce the speed of traffic entering the intersection.
    (c) To divert traffic from obstacles and expedite the flow of traffic.
    (d) To segregate opposing flow of traffic in a multi-lane highway.

15. Based on '30th hour volume' for how much per cent time during the year can the designer willingly tolerate the unfavourable operating conditions?

(a) 30 (b) 29
(c) 2.5 (d) 0.33

16. A journey from work to home made by walking to the bus, travelling by bus to the station and completing the journey by train is regarded as

(a) 4 trips (b) 3 trips
(c) 2 trips (d) 1 trip

17. Which one of the folliwing is taken into consideration for computing traffic capacity per lane of the highway?

(a) Passenger cars and light vehicles
(b) Trucks and buses
(c) Two-wheelers
(d) Equivalent of passenger cars

18. Match **List-I** (Method of traffic volume counts) with **List-II** (Equipment used) and select the correct answer using the codes given below the lists:

	List-I		List-II
A.	Manual count	**1.**	Video-
B.	Combination of manual and mechanical methods	**2.**	Pneumatic tuberecorder
C.	Automatic devices	**3.**	Watch
D.	Photographic method	**4.**	Multiple pen recorder

Codes:

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

19. Match **List-I** (Type of study) with **List-II** (Data collected) and select the correct answer using the codes given below the lists:

	List-I		List-II
A.	Public transport inventory study	**1.**	Starting and destination of riders
B.	Public transport O-D study	**2.**	Available routing and scheduling
C.	Public transport usage study	**3.**	Vehicle-km, earning per km
D.	Pulic transport operator's study	**4.**	Passenger-km, IVTT, walking time, waiting time

Codes:

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

20. Match **List-I** (Type of traffic signals) with **List-II** (Advantages) and selct the correct answer using the codes given below the lists:

List-I	List-II
A. Pre-timed signals	**1.** Useful for junction of a side street having low traffic volume with a main street having heavy flow
B. Vehicle-actuated signals	**2.** Overall optimisation of traffic flow
C. Semi-vehicle actuated signals	**3.** Delay is held to a minimum, and maximum lane capacity is achieved
D. Linked traffic signals	**4.** Most successfully used in linked system

Codes:

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

21. Consider the following statements:

Assertion (A): Prohibitory signs are part of regulatory signs which give a definite negative instruction regarding movement prohibition, restriction on weight, or speed of vehicles.

Reason (R): According to the IRC Standards, prohibitory signs are of octagonal shape with side being 900 mm and with a white border and red background for the standard size.

Of these statements:

(a) both A and R are tue and R is the correct explanation of A
(b) both A and R are true but R is not a correct explanation of A
(c) A is true but R is false
(d) A is false but R is true

22. The lost time due to starting delay on a traffic signal approach is noted to be 3 seconds, the actual green time is 25 seconds and amber time is 3 seconds. How much is the effective green time?

(a) 19 sec
(b) 25 sec
(c) 27 sec
(d) 31 sec

23. Match **List-I** (Study) with **List-II** (Purpose) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Primary road system	**1.** Population distribution
B. Economic studies	**2.** Expressways
C. Engineering studies	**3.** Traffic volume
D. Road use studies	**4.** Topographic details

Codes:

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

24. Match **List-I** (Traffic survey) with **List-II** (Method) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Spot speed	**1.** By video tape
B. Traffic volume	**2.** By road side interview
C. O-D survey	**3.** By doppler radar
D. Parking survey	**4.** By pneumatic tube

Codes:

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

25. Match **List-I** (Unit) with **List-II** (Purpose) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Clover leaf interchange	**1.** Informatory sign
B. Traffic studies	**2.** Traffic control device
C. Rotary island	**3.** Traffic flow characteristics
D. Road junction approach sign	**4.** Grade-separation

Codes:

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

26. Consider the following statements related to interchanges:

1. In diamond interchange there is the possibility of illegal wrong-way turns.
2. Diamond interchange is far superior to cloverleaf design.

Which of these statements is/are correct?

(a) 1 only
(b) 2 only
(c) Both 1 and 2
(d) Neither 1 nor 2

27. Match **List-I** (Speed) with **List-II** (Application) and select the correct answer using the code given below the lists:

List-I	List-II
A. Space mean speed	**1.** Road conditions studies
B. Journey speed	**2.** Regulatory measures
C. Running speed	**3.** Traffic flow studies
D. Spot speed	**4.** Delay studies

Codes:

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

28. Match **List-I** with **List-II** and select the correct answer using the codes given below the lists:

List-I	List-II
A. Traffic volume	1. Number of vehicles occupying a unit length of road at a given instant of time.
B. Traffic Density	2. Number of vehicles passing a given point on road in a given unit of time in a given direction.
C. Traffic Regulations	3. Where all converging vehicles are forced to move in one direction around a large central traffic island.
D. Rotary Intersection	4. Rules covering all aspects of control of vehicles, drivers and all other road users.

Codes:

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

29. Which of the following factors are **not** strictly related to design of traffic rotary intersections?

1. Radius of central island
2. Weaving length
3. Ramps and interchanges
4. Acceleration lanes

Select the correct answer using the codes given below:

(a) 1 and 2 (b) 1 and 4

(c) 2 and 3 (d) 3 and 4

30. In urban transportation planning, the 'Modal Split' is the process of which one of the following?

(a) Staggering of working hours

(b) Segregation of fast and slow modes

(c) Separation of traffic streams by flyovers

(d) Deciding the choice for a mode

31. Which one of the following geometric features requires the magnitudes of weaving angle and weaving distance for its design?

(a) Rotary design (b) Right-angle intersection

(c) Round about (d) Grade-separated junction

32. Which set of traffic studies is needed for functional design as well as for 'highway capacity' design?

(a) Origin and destination studies (b) Parking and accident studies

(c) Speed and volume studies (d) Axle load studies

33. Which one of the following traffic survey schemes is most-relevant when deciding on locating major 'routes' in a city?
(a) Traffic volume survey (b) Origin and destination survey
(c) Speed survey (d) Traffic capacity survey

34. Which one of the following equipments is useful in determining spot speed in traffic engineering?
(a) Enoscope (b) Periscope
(c) Radar (d) Tachometer

35. Which one of the following traffic signal systems is useful when there is continuous operation of group of vehicles along the main road?
(a) Simultaneous system (b) Alternate system
(c) Simple progressive system (d) Flexible progressive system

36. Match **List-I** (Type of survey) with **List-II** (Method/Instrument) and select the correct answer using the codes given below the lists:

List-I	**List-II**
A. Traffic volume dstudy	**1.** Workspot interview method
B. Speed and delay study	**2.** Doppler radar
C. Spot-speed study	**3.** Floating car method
D. Multiple character studies	**4.** Automatic vehicle counter and classifer
	5. Electronic detector

Codes:

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

37. Light reflecting devices used to guide the driver along the proper alignment are called
(a) rumble strips (b) delineators
(c) attenuators (d) litter bin

38. **Assertion (A):** In an intersection design, the relative speed is dependent on the absolute speed of intersecting vehicles and the angles between them.
Reason (R): When the angle of merging is small, the relative speed will be high.

39. Traffic flow equation for a section of road is u = 80 – 0.7k where 'u' is the speed in kmph and k is the density in vpkm (vehicles per km). The maximum expected flow is
(a) 4572 vph (b) 2286 vph
(c) 1143 vph (d) 572 vph

40. During measurement in spot speed study, a total of 1000 vehicles were observed. 85 percentile and 15 percentile speeds were obtained as 40 kmph and 10 kmph respectively. The number of

vehicles moving between speeds of 10 kmph and 40 kmph would be

(a) 350 (b) 500
(c) 600 (d) 700

41. Consider the following statements:

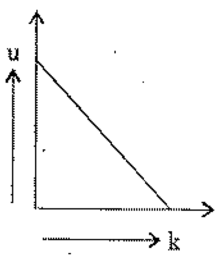
Assertion (A): If the highway passes by the town and not through it, its usefulness increases when the town is not the destination.

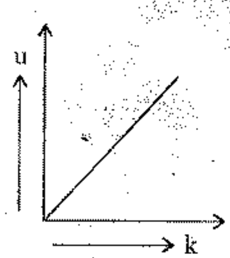
Reason (R): Potential for expansion in the area surrounding the highway increases if the highway passes by the town and not through the town.

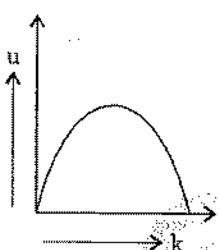
Of the statements:

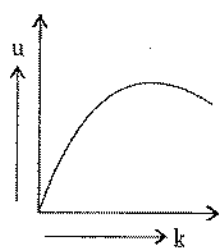
(a) both A and R are individually true and R is the correct explanation A
(b) both A and R are individually true but are R is not the correct explanation A
(c) A is true but R is false
(d) A is false but R is true

42. Which one of the following diagrams illustrates the relation between speed 'u' and density 'k' of traffic flow?

(a) 

(b) 

(c) 

(d) 

43. In speed and delay study, if the average journey time on a stretch of road length of 3.5 km is 7.55 minutes and the average stopped delay is 1.8 minutes, the average running speed will be, nearly

(a) 36.5 kmph (b) 37.5 kmph
(c) 38.5 kmph (d) 39.5 kmph

44. Consider the following statements:

Assertion (A): The peak value of the theoretical maximum capacity of the traffic flow is reached at an optimum speed.

Reason (R): As the speed of the traffic flow is increased above the optimum value, the maximum capacity of the lane starts decreasing due to increase in headway at the speed range.

Of the statements:

(a) both A and R are individually true and R is the correct explanation A
(b) both A and R are individually true but are R is not the correct explanation A
(c) A is true but R is false

(d) A is false but R is true

45. Which one of the following methods of O-D traffic surveys is conducted for comprehensive analysis of traffic and transportation data?
(a) Home interview (b) Roadside interview
(c) Registration number method (d) Postcard method

46. The lost time due to starting delay on a traffic signal is noted to be 3 s, the actual green time is 25 s and yellow time is 3 s. How much is the effective green time?
(a) 31 s (b) 28 s
(c) 25 s (d) 22 s

47. Consider the following statements:
Traffic volume data can be presented in the form of
1. annual average daily traffic
2. trend charts
3. traffic flow maps along the routes
4. modal averages
Which of these statements are correct?
(a) 1, 2 and 3 (b) 1, 2 and 4
(c) 1 and 2 (d) 3 and 4

48. If L is the length of vehicle is meters, C is the clear distance between two consecutive vehicles (stopping sight distance), V is the speed of vehicles in km/hour; then the maximum number (N) of vehicles/hour is equal to

(a) $N = \dfrac{1000V}{C+L}$ (b) $N = \dfrac{C+L}{1000V}$

(c) $N = \dfrac{1000V}{C-L}$ (d) $N = \dfrac{1000V}{L+V}$

49. Weaving length is the distance
(a) equal to half the perimeter of central rotary
(b) between the channelizing islands
(c) equal to total width of adjoining radial roads
(d) equal to diameter of central rotary

50. When the speed of the traffic flow becomes zero, then
(a) traffic density attains its maximum value whereas traffic volume becomes zero
(b) traffic density and traffic volume both attain respective maximum values
(c) traffic density and traffic volume both become zero
(d) traffic density becomes zero whereas traffic volume attains its maximum value

51. Maximum service volume of a road is defined as the total number of vehicles that
(a) can pass a given point in a specified period of time
(b) can be accommodated on a unit length of road
(c) can pass a given point in unit time
(d) None of these

52. For the movement of vehicles at an intersection of two roads without any interference, which type of grade separation is generally preferred?
(a) Delta
(b) Diamond
(c) Trumpet
(d) Cloverleaf

53. In highway geometric design, cumulative speed distribution is drawn and the design is checked at which percentile?
(a) 85th percentile
(b) 95th percentile
(c) 100th percentil
(d) 98th percentile

54. Match **List-I** (Terms related to traffic signal) with **List-II** (Description of terms used in traffic signal) and selct the correct answer using the codes given below the lists:

List-I	List-II
A. Load factor	**1.** Part of signal cycle allocated to traffic movement
B. Phase	**2.** Quantitative assessment usually measured by queue length; vehicle delay, etc.
C. Level of service	**3.** Measure of degree of utilisation of an inter-sector approach road

Codes:

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

55. **Assertion (A):** PCU values are used to calculate the total equivalent traffic volume on road.
Reason (R): PCU value of any vehicle is an indicator of relative damage caused by a vehicle to a pavement as compared to passenger car.

56. **Assertion (A):** Point of potential conflicts in case of both roads of intersection having two lanes, two-way traffic are 24.
Reason (R): Point of potential conflicts are not dependent on the number of lanes on intersecting lanes.

57. **Assertion (A):** The traffic rotary is an effective traffic management system if the traffic volume exceeds 5000 vehicles/hour.
Reason (R): In rotary merging, weaving and diverging improve traffic management.

58. Which one of the following represents the basic capacity of a single lane road?
(a) $\frac{1000s}{v}$
(b) $\frac{1000v}{s}$
(c) $\frac{1000h}{v}$
(d) $\frac{1000v}{h}$

59. What is the maximum number of passenger cars that can pass a given point on a lane or roadway during one hour under ideal roadway and traffic conditions, known as?
(a) Practical capacity
(b) Possible capacity
(c) Basic capacity
(d) Road capacity

60. For which one of the following volumes, are the highway facilities generally designed?

(a) Average daily traffic
(b) Annual average hourly volume
(c) Peak hourly volume
(d) 30th highest hourly volume

61. Consider the following statements with respect to traffic capacity studies:

1. The value of possible capacity varies from zero to basic capacity.
2. Basic capacity and possible capacity represent two extreme cases of road ways and traffic conditions.

Which of these statements is/are correct?

(a) 1 only
(b) 2 only
(c) Both 1 and 2
(d) Neither 1 nor 2

62. Which one of the following holds good for the maximum number of vehicles that can pass a given point on a lane or roadway during one hour under prevailing roadway and traffic conditions?

(a) Basic capacity
(b) Possible capacity
(c) Practical capacity
(d) Theoretical capacity

63. Match **List-I** (Item) with **List-II** (Definition) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Basic capacity	**1.** Vehicles that pass a given point on a lane/hour
B. Traffic density	**2.** Maximum number of vehicles that can pass a given point on a lane during one hour under prevailing roadway and traffic conditions
C. Traffic volume	**3.** Number of vehicles occupying a unit length of a lane at a given instant
D. Possible capacity	**4.** Maximum number of vehicles that can pass a given point on any lane during one hour under ideal roadway and traffic conditions

Codes:

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

64. How many number of points of conflicts can rise with one-way regulation in both directions on an intersection having 4 legs?

(a) 4
(b) 6
(c) 8
(d) 10

65. How is a road junction designated when it achieves the purpose that traffic streams are divided to enable them to pass over or under each other?

(a) Subway
(b) Loop road
(c) Flyover
(d) By-pass road

66. Which one of the following correctly represents time-mean speed?

(a) Average speed of vehicles in a certain road length at any time.
(b) Speed distribution of vehicles at a point on the roadway.
(c) Average speed maintained by vehicles over a particular stretch of road.
(d) Speed obtained by dividing the total distance travelled by the total time taken.

67. The speed and delay studies on a defined section of highway are conducted by
(a) radar gun (b) traffic counters
(c) moving car method (d) enoscope

68. The road geometrics in India are designed for the
(a) 98^{th} highest hourly traffic volume (b) 85^{th} highest hourly traffic volume
(c) 50^{th} highest hourly traffic volume (d) 30^{th} highest hourly traffic volume

69. A transport company operates a scheduled daily truck service between city P and city Q. One-way journey time between these two cities is 85 hours. A minimum layover time of 5 hours is to be provided at each city. How many trucks are required to provide this service?
(a) 4 (b) 6
(c) 7 (d) 8

70. A single lane unidirectional highway has a design speed of 65 kmph. The perception-brake-reaction time of drivers is 2.5 seconds and the average length of vehicles is 5m. The coefficient of longitudinal friction of the pavement is 0.4. The capacity of this road in terms of 'vehicles per hour per lane' is
(a) 1440 (b) 750
(c) 710 (d) 680

71. Name the traffic survey data which is plotted by means of **Desire lines**.
(a) Accident (b) Classified volume
(c) Origin and Destination (d) Speed and Delay

72. For designing a 2-phase fixed type signal at an intersection having North-South and East-West road where only straight ahead traffic is permitted, the folowing data is available.

Parameter	North	South	East	West
Design Hour Flow (PCU/hr)	1000	700	900	550
Saturation Flow (PCU/hr)	2500	2500	3000	3000

Total time lost per cycle is 12 seconds. The cycle length (seconds) as per Webster's approach is
(a) 67 (b) 77
(c) 87 (d) 91

73. On an urban road, the free mean speed was measured as 70 kmph and the average spacing between the vehicles under jam condition as 7.0 m. The speed-flow-density equation is given by:

$$U = U_{sf}\left[1 - \frac{k}{k_j}\right] \text{ and } q = Uk$$

where,
U = space-mean speed (kmph);
U_{sf} = free mean speed (kmph);
k = density (veh/km);

k_j = jam density (veh/km);

q = flow (veh/hr)

The maximum flow (veh/hr) per lane for this condition is equal to

(a) 2000 (b) 2500

(c) 3000 (d) None of the above

74. If a two-lane National Highway and a two-lane State Highway intersect at right angles, the number of potential conflict points at the intersection, assuming that both the roads are two-way is

(a) 11 (b) 17

(c) 24 (d) 32

75. In signal design as per Indian Roads Congress specifications, if the sum of the ratios of normal flows to saturation flow of two directional traffic flow is 0.50 and the total lost time per cycle is 10 seconds, the optimum cycle length in seconds is

(a) 100 (b) 80

(c) 60 (d) 40

76. The capacities of "One-way 1.5 m wide sidewalk (persons per hour)" and "One-way 2-lane urban road (PCU per hour, with no frontage access, no standing vehicles and very little cross traffic)" are respectively

(a) 1200 and 2400 (b) 1800 and 2000

(c) 1200 and 1500 (d) 2000 and 1200

77. The shape of the STOP sign according to **IRC:67-2001** is

(a) circular (b) triangular

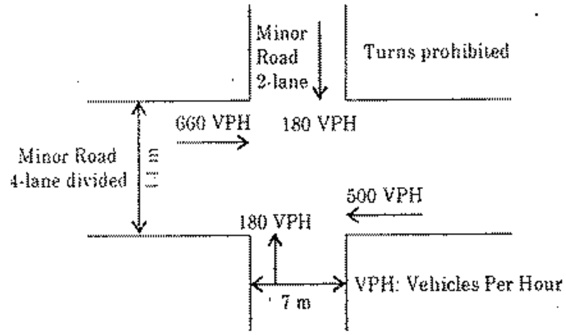
(c) octagonal (d) rectangular

78. A round about is provided with an average entry width of 8.4 m, width of weaving section as 14 m, and length of the weaving section between channelizing islands as 35 m. The crossing traffic and total traffic on the weaving section are 1000 and 2000 PCU per hour respectively. The nearest rounded capacity of the roundabout (in PCU) per hour) is

(a) 3300 (b) 3700

(c) 4500 (d) 5200

79. Design parameters for a signalized intersection are shown in the figure below. The green time calculated for major and minor roads are 34 and 18 s, respectively.



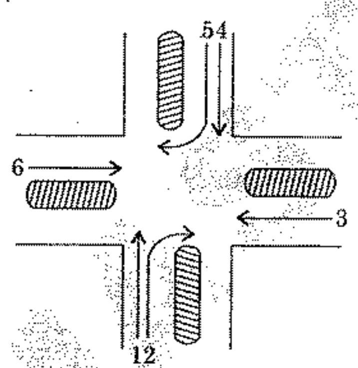
The critical lane volume on the major road changes to 440 vehicles per hour per lane and the critical lane volume on the minor road remains unchanged. The green time will

(a) increase for the major road and remain same for the minor road.
(b) increase for the major road and decrease for the minor road.
(c) decrease for both the roads.
(d) remain unchanged for both the roads.

80. On a specific highway, the speed-density relationship follows the Greenberg's model [$v = v_f \log_r(k_j/k)$], where v_f and k_j are the free flow speed and jam density respectively. When the highway is operating at capacity, the density obtained as per this model is

(a) ek_j
(b) k_j
(c) $\frac{k_j}{2}$
(d) $\frac{k_j}{e}$

81. A three-phase traffic signal at an intersection is designed for flows shown in the figure below. There are six groups of flows identified by the numbers 1 through 6. Among these 1, 3, 4 and 6 are through flows and 2 and 5 are right turning. Which phasing scheme is **not feasible**?



Combination choice	Phase I	Phase II	Phase III
P	1, 4	2, 5	3, 6
Q	1, 2	4, 5	3, 6
R	2, 5	1, 3	4, 6
S	1, 4	2, 6	3, 5

(a) P
(b) Q
(c) R
(d) S

ANSWERS

1.	(c)	2.	(a)	3.	(c)	4.	(a)	5.	(b)	6.	(b)	7.	(b)
8.	(b)	9.	(c)	10.	(c)	11.	(d)	12.	(c)	13.	(c)	14.	(d)
15.	(d)	16.	(b)	17.	(d)	18.	(b)	19.	(d)	20.	(a)	21.	(c)
22.	(b)	23.	(a)	24.	(c)	25.	(a)	26.	(a)	27.	(c)	28.	(c)
29.	(d)	30.	(d)	31.	(a)	32.	(c)	33.	(b)	34.	(a)	35.	(c)
36.	(a)	37.	(b)	38.	(c)	39.	(b)	40.	(d)	41.	(c)	42.	(a)
43.	(a)	44.	(a)	45.	(b)	46.	(c)	47.	(c)	48.	(a)	49.	(b)
50.	(a)	51.	(c)	52.	(d)	53.	(d)	54.	(b)	55.	(c)	56.	(c)
57.	(d)	58.	(b)	59.	(c)	60.	(d)	61.	(c)	62.	(c)	63.	(d)
64.	(b)	65.	(c)	66.	(b)	67.	(c)	68.	(d)	69.	(d)	70.	(c)
71.	(c)	72.	(b)	73.	(b)	74.	(c)	75.	(d)	76.	(a)	77.	(c)
78.	(b)	79.	(a)	80.	(d)	81.	(c)						

HINTS AND SOLUTIONS

1. (c).

Sol. Desire lines are graphical representation of O & D survey. These are straight lines connecting the origin points with destinations, summarized into different area groups. The widths of such desire lines is drawn proportional to the number of trips in both direction.

4.(a)

Sol. $u = (55 - 0.44\,k)$

Flow $q = ku$

$= k\,(55 - 0.44k)$

$= (55\,k - 0.44\,k^2)$

For maximum Flow =

$$\left(\frac{dq}{dk}\right) = 0 = (55 - 0.88\,k)$$

$$k = \left(\frac{55}{0.88}\right) \quad ...(i)$$

$$q_{max} = \frac{55}{0.88}\left(55 - \frac{0.44\times55}{0.88}\right)$$

$= 1718$ VPh

7.(b).

Sol. Free Speed $= 80$ km/hr

Density $k = 70$ V/km

Maximum Flow q_{max} =

$$\left(\frac{V\times k}{4}\right) = \frac{80\times70}{4} = 1400\,V/hr$$

8. (b).

Sol. $q_1 = 500$ PCV/hr, $S_1 = 1600$ PCV/hr

$q_2 = 300$ PCV/hr, $S_2 = 1600$ PCU/hr

$$y_1 = \left(\frac{q_1}{S_1}\right) = \left(\frac{500}{1600}\right) = \left(\frac{5}{16}\right)$$

$$y_2 = \left(\frac{q_2}{S_2}\right) = \left(\frac{300}{1600}\right) = \left(\frac{3}{16}\right)$$

$$y = (y_1 + y_2) = \left(\frac{5}{16} + \frac{3}{16}\right) = 0.5$$

$$L = (2n + R) = 16$$

$$\therefore \quad C_0 = \frac{(1.5L+5)}{(1-y)} = \frac{(1.5\times16+5)}{(1-0.5)}$$

$$= \left(\frac{24+5}{0.5}\right) = 58\,\text{sec}$$

9. (c)

Sol. Two road with two lane. two way traffic, the total number of potential major conflict points = 16.

12.

Sol. We know that

$$q = \frac{(m_a + m_w)}{(t_a + t_\omega)} \quad ...(i)$$

$n_a = 303$ vehicle

$n_y = 17$ vehicle

$$ta = t_\omega = \frac{5}{70}$$

$$\therefore \quad q = \frac{(17+303)}{\left(\frac{5}{10} + \frac{5}{70}\right)} = 2240\,\text{VPh}$$

13.(c)

Sol. Monthly Expansion Factor $= \dfrac{650\times7}{3625} = 1.25$

15.(d)

Sol. Only for 29 hours in a year the traffic volume exceeds the design hourly volume

Therefore % time $= \dfrac{29\times100}{(365\times24)} = 0.33$

16. (b)

Sol. Trip is defined as the one way movement houring single purpose and mode of travel between a point of origin and a point of destination.

22.

Sol. Effective green time

$(Ge) = (G + S - L)$

$= 25 + 3 - (3 + 3) = 22$ sec.

$L = (L_i + L_f)$

$L = (3 + 3) = 6$ sec. (In absence of any data on the final lost time. It can be taken to be equal to amber time)

39. (b). Flow is givenby $q = ku$

$q = k\,(80 - 0.7k)$

$q \quad (80k - 0.7k^2) \quad ...(i)$

For maximum Flow $= \left(\dfrac{dq}{dk}\right)$

$= 0 = (80 - 1.4k)$

$$\Rightarrow \quad k = \left(\frac{80}{1.4}\right)$$

$$q_{max} = \frac{80}{1.4}\left[80 - 0.7 \times \frac{80}{1.4}\right]$$

$$= \left(\frac{3200}{1.4}\right) = 2286\text{VPh}$$

42. (a).

Sol. The relation between speedn u and jam density k is given by $u = (a - bk)$

63. (d)

Sol. (i) *Basic Capacity* is defined as the maximum capacity possible on a road under most ideal condition is called basic capacity.

(ii) *Possible Capacity* is defined as the actual value of volumes that may be possible under general condition of traffic is called principle capacity. possible capacity may vary from zero to basic capacity.

64.

Sol. For one way regulation in both direction on anintersection having 4-legs,

Crossing conflict = 4

Merging conflicts = 2

Total = 6

66.

Sol. *Space mean speed* represents the average speed of vehicle in a certain road length at any time.

Time mean speed represents the speed distribution of vehicle at a point on the roadway.

70. (c)

$$\textbf{Sol.} S = \left(0.278\,Vt + \frac{(V \times 0.278)^2}{2gf} + L\right)$$

$$= (0.278 \times 65 \times 2.5) + \frac{(0.278 \times 65)^2}{2 \times 9.81 \times 0.4} + 5 = 91.78\text{ m}$$

$$C = \frac{1000V}{S} = \frac{1000 \times 65}{(91.78)} = 708.2 = 710\text{VPhr}$$

72. (b)

Sol. Only straight ahead traffic is permitted

In the N-S section, higher value is 1000 and saturation value is 2500

$$y_1 = \frac{1000}{2500} = 0.4$$

In the E-W section

higher value = 900

saturation value = 3000

$$y_1 = \frac{900}{3000} = 0.3$$

$$y = y_1 + y_2$$

$$= 0.4 + 0.3 = 0.7$$

Time lost per cycle $L = 12s$

$$\text{Optimum cycle time} = \frac{1.5L + 5}{1 - y}$$

$$= \frac{1.5 \times 12 + 5}{1 - 0.7} = 76.67s = 77\text{ s}$$

73. (b).

$$\textbf{Sol.} \left[q_{max} = \left(\frac{v_{sf} \times k}{4}\right) = \left(\frac{70 \times 1000}{7 \times 4}\right) = 2500\text{ Veh / hr}\right.$$

75. (d).

$$\textbf{Sol.} \quad C_0 = \frac{1.5L + 5}{1 - Y} = \frac{(1.5 \times 10 + 5)}{(1 - 0.5)} = 40\text{ sec.}$$

78. (b)

$$\textbf{Sol.} \quad Q_P = \frac{280\,W\left(1 + \frac{e}{W}\right)\left(1 - \frac{p}{3}\right)}{\left(1 + \frac{W}{L}\right)}$$

$$= \frac{280 \times 14\left(1 + \frac{8.4}{14}\right)\left(1 - \frac{0.5}{3}\right)}{\left(1 + \frac{14}{35}\right)} = \mathbf{3733}$$

80. (d)

$$\textbf{Sol.} \quad V = V_f \log_e\left(\frac{K_j}{K}\right)$$

$$q = KV = KV_f \log_e\left(\frac{K_j}{K}\right)$$

For capacity

$$\frac{dq}{dK} = V_f\left[K \times \frac{K}{K_j}\left(-\frac{K_j}{K^2}\right) + \log\left(\frac{K_j}{K}\right) \times 1\right]$$

$$0 = -1 + \log_e\left(\frac{K_j}{K}\right)$$

$$\Rightarrow \quad \frac{K_j}{K} = -e \qquad \therefore \qquad K = \left(\frac{K_j}{e}\right)$$

CHAPTER

4

Highway Materials

PAVEMENT MATERIALS: SOIL

Sub grade soil

- Soil is an accumulation or deposit of earth material, derived naturally from the disintegration of rocks or decay of vegetation, that can be excavated readily with power equipment in the field or disintegrated by gentle mechanical means in the laboratory.
- The supporting soil beneath pavement and its special under courses is called sub grade.
- Compacted sub grade is the soil compacted by controlled movement of heavy compactors.

Desirable properties of sub grade soil

The desirable properties of sub grade soil as a highway material are

(1) Stability
(2) Incompressibility
(3) Permanency of strength
(4) Minimum changes in volume and stability under adverse conditions of weather and ground water
(5) Good drainage, and
(6) Ease of compaction

The tests used to evaluate the strength properties of soils may be broadly divided into three groups:

(1) Shear tests (2) Bearing tests (3) Penetration test

- *Shear tests* are usually carried out on relatively small soil samples in the laboratory. In order to find out the strength properties of soil, a number of representative samples from different locations are tested. Some of the commonly known shear tests are direct shear test, triaxial compression test, and unconfined compression test.
- *Bearing tests* are loading tests carried out on sub grade soils in-situ with a load bearing area. The results of the bearing tests are influenced by variations in the soil properties within the stressed soil mass underneath and hence the overall stability of the part of the soil mass stressed could be studied.
- *Penetration tests* may be considered as small scale bearing tests in which the size of the loaded area is relatively much smaller and ratio of the penetration to the size of the loaded area is much greater than the ratios in bearing tests. The penetration tests are carried out in the field or in the laboratory.

CALIFORNIA BEARING RATIO TEST

- California Bearing Ratio (CBR) test was developed by the California Division of Highway as a method of classifying and evaluating soil-sub grade and base course materials for flexible

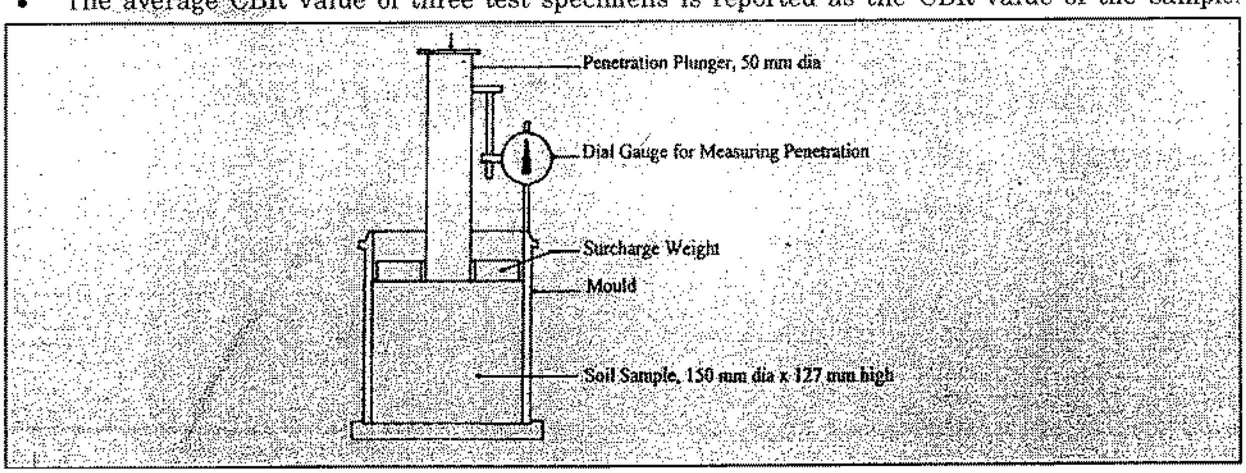
- CBR test, an empirical test, has been used to determine the material properties for pavement design.
- Empirical tests measure the strength of the material and are not a true representation of the resilient modulus.
- It is a penetration test wherein a standard piston, having an area of 19.62 cm^2 (or 50 mm dia.), is used to penetrate the soil at a std. rate of 1.25 mm/min. The pressure up to a penetration of 12.5 mm and its ratio to the bearing value of a std. crushed rock is termed as the CBR.
- In most cases, CBR decreases as the penetration increases.
- The ratio at 2.5 mm penetration is used as the CBR.
- In some case, the ratio at 5 mm may be greater than that at 2.5 mm. If this occurs, the ratio at 5 mm should be used if confirmed by repeating the test.
- The CBR is a measure of resistance of a material to penetration of standard plunger under controlled density and moisture condition. The test may be conducted in re-moulded or undisturbed specimen in laboratory. It is extensively used for field correlation of flexible pavement thickness requirements.

Test Procedure

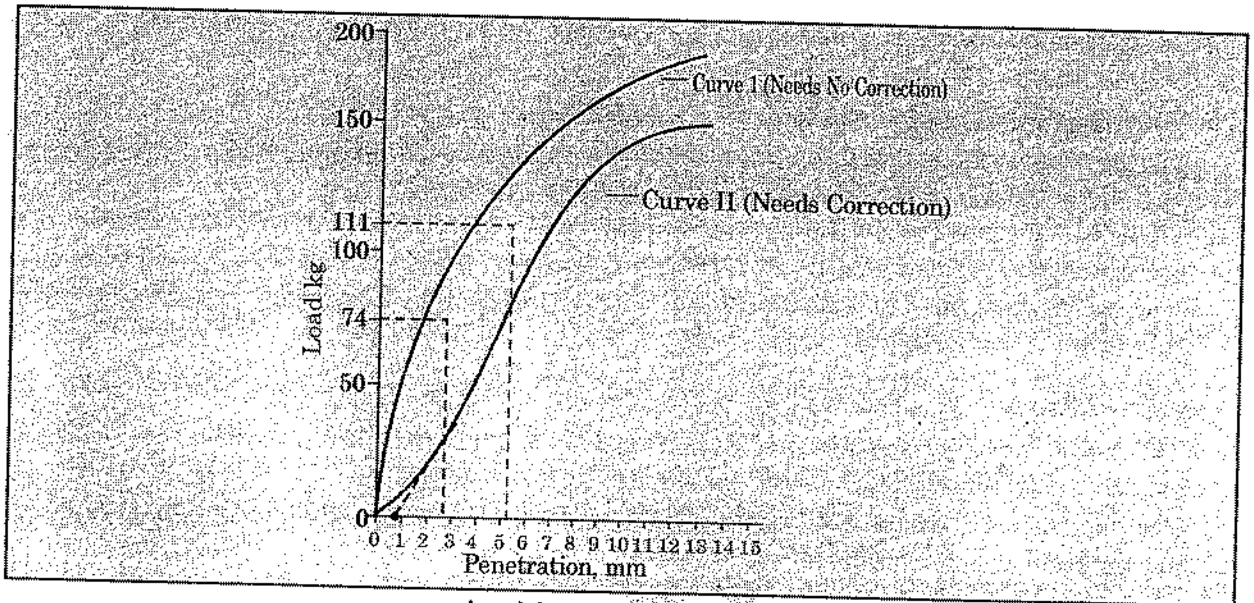
- The laboratory CBR apparatus consists of a mould 150 mm dia with a base plate and a collar, a loading frame & dial gauges for measuring the penetration values and the expansion on soaking.
- The specimen in the mould is soaked in water for four days and the swelling and water absorption values are noted.
- Load is applied on the sample by a standard plunger with dia of 50 mm at the rate of 1.25 mm/min. A load penetration curve is drawn.
- The load values on standard crushed stones are 1370 kg (70 kg/cm^2) and 2055 kg (105 kg/cm^2) at 2.5 mm and 5.0 mm penetrations respectively.
- CBR value is expressed as a percentage of the actual load causing the penetrations of 2.5 mm or 5.0 mm to the standard loads mentioned above. Therefore,

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

- Two values of CBR will be obtained. If the value of 2.5 mm is greater than that of 5.0 mm penetration, the former is adopted.
- If the CBR value obtained from test at 5.0 mm penetration is higher than that at 2.5 mm, then the test is to be repeated for checking.
- If the check test again gives similar results, then higher value obtained at 5.0 mm penetration is reported as the CBR value.
- The average CBR value of three test specimens is reported as the CBR value of the sample.



- If the specimen has surface irregularities, the initial portion of the curve may have concavity upwards. In that case, a tangent is drawn to the curve at the point of greatest slope (Curve in figure below) This tangent plus the convex portion of the original curve is the corrected curve, with the origin moved to the point where the tangent cuts the X-axis.



Load vs penetration Curve

Typical CBR Values of Soils and their Rating

Soil	Range of CBR values	Rating
Clay	2–5	Very poor subgrade
Silt	5–8	Poor subgrade
Sand	8–20	Fair to good subgrade
Gravel	20–30	Excellent subgrade
	30–60	Good sub-base
	60–80	Good base
	80–100	Best Base

CBR values have been roughly correlated to Modulus of Subgrade Reaction Table .

Correlation Between CBR and k Value

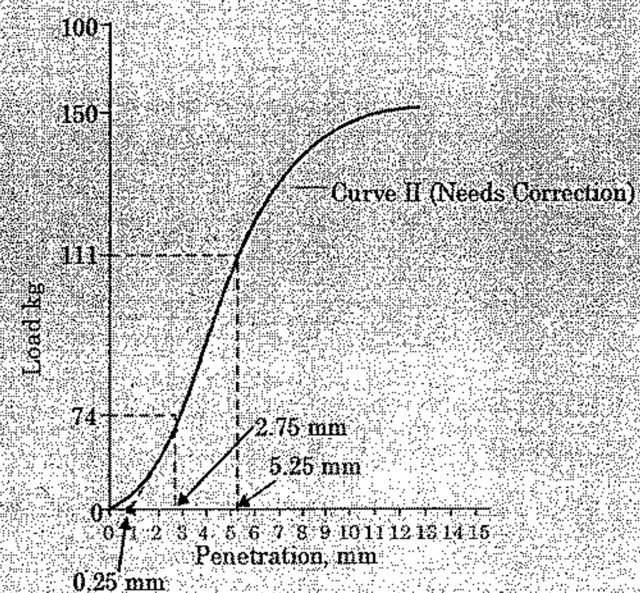
CBR	2	3	4	5	7	10	20	50	100
k value MN/m²	20	27	34	41	48	54	68	136	218

Example 1

Load and penetration values from a CBR test are given below. Calculate the CBR value.

Penetration	0	0.5	1.0	1.5	2.0	2.5	3	4	5	7.5	10	12.5
Load (kg)	0	7	24	41	59	70	81	98	110	129	143	150

Sol.



The load-penetration curve is plotted in Fig. (Curve II). The observation need correction. The tangent to the curve meets the x-axis at the value of 0.25 mm. The origin needs shifting by this amount. Thus, the loads at 2.5 mm and 5.00 mm penetration are 74 and 111 kg respectively.

$$\text{CBR value from 2.5 mm penetration} = \frac{74}{1370}\times 100 = 5.4$$

$$\text{CBR value from 5.00 mm penetration} = \frac{111}{2055}\times 10 = 5.4$$

The CBR value is 5.4 per cent.

PLATE BEARING TEST

- Plate bearing test is used to evaluate the support capability of sub-grades, bases and in some cases, complete pavement.
- Data from the tests are applicable for the design of both flexible and rigid pavements.
- In plate bearing test, a compressive stress is applied to the soil or pavement layer through rigid plates relatively large size and the deflections are measured for various stress values.

Test Procedure

- The test site is prepared and loose material is removed so that the 75 cm diameter plate rests horizontally in full contact with the soil sub-grade. The plate is seated accurately and then a seating load equivalent to a pressure of 0.07 kg/cm^2 (320 kg for 75 cm diameter plate) is applied and released after a few seconds. The settlement dial gauge is now set corresponding to zero load.
- A load is applied by means of jack, sufficient to cause an average settlement of about 0.25 cm. When there is no perceptible increase in settlement or when the rate of settlement is less than 0.025 mm per minute (in the case of soils with high moisture content or in clayey soils) the load dial reading and the settlement dial readings are noted.
- Deflection of the plate is measured by means of deflection dials; placed usually at one-third points of the plate near it's outer edge.

- To minimise bending, a series of stacked plates should be used.
- Average of three or four settlement dial readings is taken as the settlement of the plate corresponding to the applied load. Load is then increased till the average settlement increase to a further amount of about 0.25 mm, and the load and average settlement readings are noted as before. The procedure is repeated till the settlement is about 1.75 mm or more.
- Allowance for worst subgrade moisture and correction for small plate size should be dealt properly. For worst moisutre condition of subgrade an unsoaked & other soaked sample of soil is taken and pressure required to cause settlement of 0.125 cm is noted in both. Hence

$$K_{soaked} = \frac{K_{unsoaked}}{P_{unsoaked}} \times P_{soaked}$$

K_{soaked} = Modulus of subgrade reaction for soaked condition.

$K_{unsoaked}$ = Modulus of subgrade reaction for unsoaked condition.

P_{soaked} = Pressure required to produce 0.125 cm settlement in soaked condition.

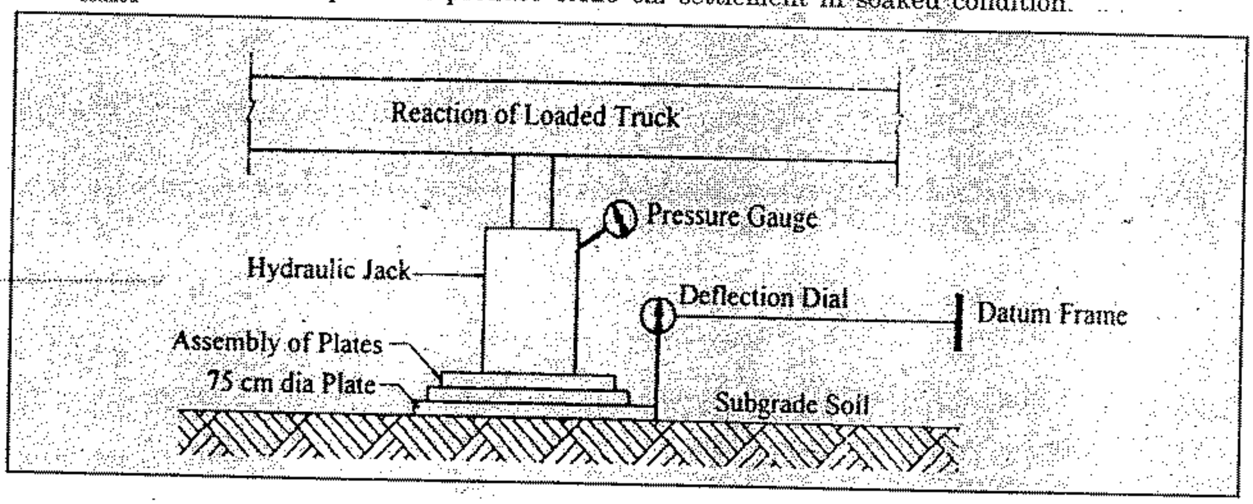
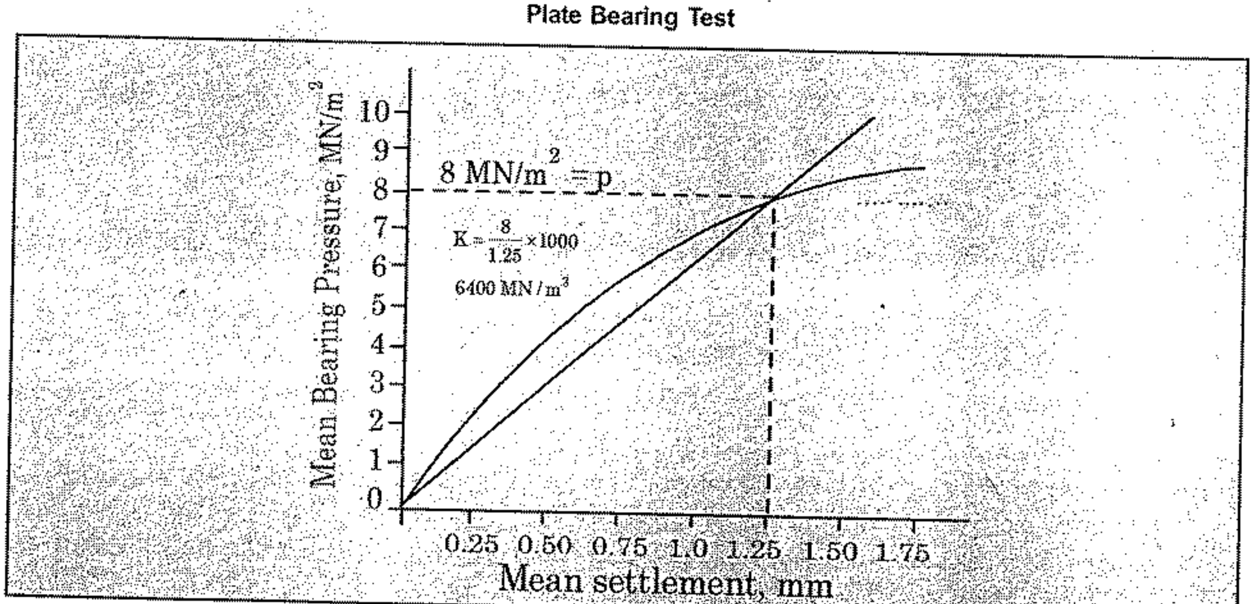


Plate Bearing Test



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

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

- The load-settlement curve figure should theoretically be a straight line, and its slope should give the value of k. But it is usual to get a curved relationship, in which case k is determined by drawing a straight line through the origin and a point on the curve corresponding to a settlement of 1.25 mm

Then, k = loading pressure corresponding to a settlement of 1.25 mm/ 1.25 = $\frac{p}{1.25}$

For the conditions given in Figure

$$K = \frac{8 \text{MN/m}^2}{1.25 \times 10^{-3}} = 6400 \text{MN/m}^3$$

- If a 30 cm dia plate is used, a rough correlation between the k value obtained using 75 cm diameter plate and the k value obtained using a 30 cm dia plate is given by:
  $k_{75} = 0.5\ k_{30}$
- A theoretical relationship can also be derived to deal with problems of the above nature. When a rigid circular plate is loaded on a soil of elastic property, the settlement is given by :

$$\Delta = \frac{1.18 pa}{E}$$

Since, $$k = \frac{p}{\Delta},$$

$k = \frac{E}{1.18.a}$, E being Modulus of elasticity of the plate and a being the diameter of the plate.

It thus follows that : $k_1 a_1 = k_2 a_2 = \frac{E}{1.18}$, constant,

where, the subscripts 1 and 2 refer to values with tests with plates of different diameter.

Then, $$\boxed{k_{75} = k_{30} \times \frac{30}{75} = 0.4\, k_{30}}$$

Example 2

Plate bearing tests conducted on a 30 cm dia plate yielded the following observations:

Load	270	580	770	1010	1260	1480	1690
Settlement (mm)	0.25	0.50	0.75	1.00	1.25	1.50	1.75

Determine the k value corresponding to a plate of 75 cm diameter.

Sol. At a settlement of 1.25 mm, load = 1260 kg.

$$\text{Loading stress, } p = \frac{1260}{0.7854 \times 30^2} \times \frac{9.81}{10^6} = 0.175 \text{ MN/m}^2 \text{/m}$$

$$k = \frac{P}{1.25} \times 1000 \text{MN/m}^2 \text{/m} = 140 \text{ MN/m}^2 \text{/m}$$

$$k_{75} = 0.4\ k_{30} = 0.4 \times 140 = 56 \text{ MN/m}^2 \text{/m}$$

PAVEMENT MATERIALS : AGGREGATES

- Aggregate is a collective term for the mineral materials such as sand, gravel, and crushed stone that are used with a binding medium (such as water, Portland cement, lime, etc.) to form compound materials (such as bituminous concrete and Portland cement concrete).
- By volume, aggregate generally accounts for 92 to 96 percent of Bituminous concrete and about 70 to 80 percent of Portland cement concrete.

DESIRABLE PROPERTIES

Strength

- The aggregates used in top layers are subjected to stress action due to traffic wheel load, wear and tear, crushing.
- For a high quality pavement, the aggregates should possess high resistance to crushing, and to withstand the stresses due to traffic wheel load.

Hardness

- The aggregates used in the surface course are subjected to constant rubbing or abrasion due to moving traffic.
- The aggregates should be hard enough to resist the abrasive action caused by the traffic movement.
- The abrasive action is severe when steel tyred vehicles moves over the aggregates exposed at the top surface.

Toughness

- Resistance of the aggregates to impact is termed as toughness.
- Aggregates used in the pavement should be able to resist the effect caused by the jumping of the steel tyred wheels from one particle to another at different levels causes severe impact on the aggregates.

Shape of aggregates

- The flaky and elongated particles will have less strength and durability when compared with cubical, angular or rounded particles of the same aggregate.
- Hence too flaky and too much elongated aggregates should be avoided as far as possible.

Adhesion with bitumen

- The aggregates used in bituminous pavements should have less affinity with water when compared with bituminous materials, otherwise the bituminous coating on the aggregate will be stripped off in presence of water.

Durability

- The property of aggregates to withstand adverse action of weather is called soundness.
- The aggregates are subjected to the physical and chemical action of rain and bottom water, impurities there-in and that of atmosphere, hence it is desirable that the road aggregates used in the construction should be sound enough to withstand the

Freedom from deleterious particles.

- Aggregates used in bituminous mixes or portland cement concrete mixes usually require the aggregates to be clean, tough and durable in nature and free from excess amount of flat or elongated pieces, dust, clay balls and other objectionable material.

AGGREGATE TESTS

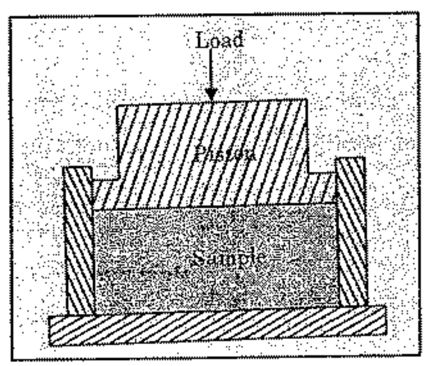
In order to decide the suitability of the aggregate for use in pavement construction, following tests are carried out:

(1) Crushing test
(2) Abrasion test
(3) Impact test
(4) Soundness test
(5) Shape test
(6) Specific gravity and water absorption test
(7) Bitumen adhesion test

Crushing test

- One of the mode in which pavement material can fail is by crushing under compressive stress.
- The aggregate crushing value provides a relative measure of resistance to crushing under gradually applied crushing load.
- The test consists of subjecting the specimen of aggregate in standard mould to a compression test under standard load conditions.
- Dry aggregates passing through 12.5mm sieves and retained on 10 mm sieves are filled in a cylindrical measure of 11.5 cm diameter and 18 cm height in three layers.
- Each layer is tampered 25 times with standard tamping rod.
- The test sample is weighed and placed in the test cylinder in three layers each layer being tampered again.
- The specimen is subjected to a compressive load of 40 tones gradually applied at the rate of 4 tones per minute.
- Crushed aggregates are then sieved though 2.36 mm sieve and weight of passing material (W2) is expressed as percentage of the weight of the total sample (W_1) which is the aggregate crushing value

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



Crushing test setup

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

Note: The crushing value for surface course should be less than 30% and should not exceed 45% for base course.

Abrasion test

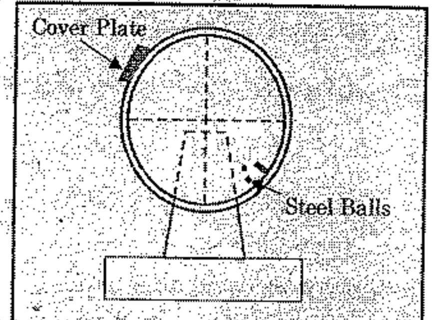
- Abrasion test is carried out to test the hardness property of aggregates and to decide whether they are suitable for different pavement construction works.

Test are of following type :

(1) Los Angeles abrasion test (2) Devel abrasion test (3) Dorry abrasion test

- Los Angeles abrasion test is a preferred one for carrying out the hardness property and has been standardized in India (IS:2386 part-IV).
- The principle of Los Angeles abrasion test is to find the percentage wear due to relative rubbing action between the aggregate and steel balls used as abrasive charge.

  Los Angeles machine consists of circular drum of internal diameter 700 mm and length 520 mm mounted on a horizontal axis enabling it to be rotated .
- An abrasive charge consisting of cast iron spherical balls of 48 mm diameters and weight 340-445g is placed in the cylinder along with the aggregates.
- The number of the abrasive spheres varies according to the grading of the sample.
- The quantity of aggregates to be used depends upon the gradation and usually ranges from 5-10 kg.
- The cylinder is then locked and rotated at the speed of 30-33 rpm for a total of 500-1000 revolutions depending upon the gradation of aggregates.
- After specified revolutions, the material is sieved through 1.7 mm sieve and passed fraction is expressed as percentage total weight of the sample.
- This value is called Los Angeles abrasion value. A maximum value of 40 percent is allowed for WBM base course in Indian conditions.



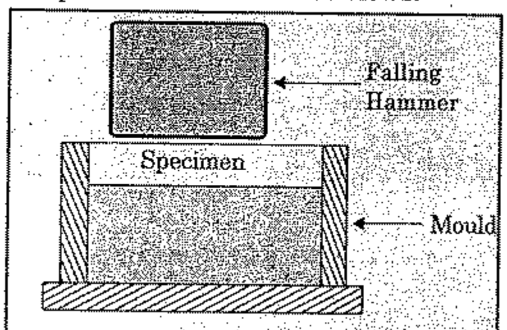
Los Angeles abrasion test setup

- For bituminous concrete, a maximum value of 35 is specified.

Impact test

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

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



Impact test setup

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

Note: The abrasion value of good aggregates for high quality pavement materials shoudl be less than 30%, however for base course in WBM it may be 50%.

Coefficient of hardness = $20 - \frac{\text{Loss of Wt. in gram}}{3}$

Soundness test

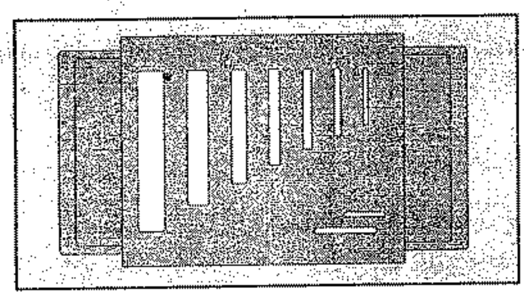
- Soundness test is intended to study the resistance of aggregates to weathering action, by conducting accelerated weathering test cycles.
- The porous aggregates subjected to freezing and thawing are likely to disintegrate prematurely. To ascertain the durability of such aggregates, they are subjected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

Shape test

The particle shape of the aggregate mass is determined by the percentage of flaky and elongated particles in it. Aggregates which are flaky or elongated are detrimental to higher workability and stability of mixes.

Flakiness Index

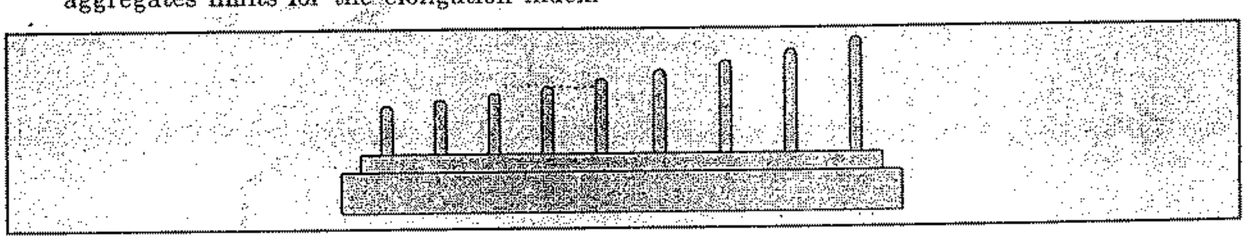
- The flakiness index is defined as the percentage by weight of aggregate particles whose least dimension is less than 0.6 times their mean size.



Flakiness gauge

Elongation Index

- The elongation index of an aggregate is defined as the percentage by weight of particles whose greatest dimension (length) is 1.8 times their mean dimension. This test is applicable to aggregates limits for the elongation index.



Elongation gauge

Angularity Number

- This represents the degree of packing
- $\boxed{\text{Angularity no} = 67 - \%\ \text{solid volume.}}$
- 67 represents the volume of solids (in %) of most rounded gravels in a well compacted state, which would have 33% voids. Thus the angularity no measures the voids in excess of 33%.
- Higher the angularity number, more angular is the aggregate.

- The range of angularity No. for aggregates used for construction is 0 to 11.

  Angularity No. $= 67 - \dfrac{100W}{C \times G_a}$ [The value is expressed as nearest whole number]

  where, W = weight of aggregate in a cylinder ; C = Weight of water in same cylinder
  G_a = Sp. gravity of aggregate.

Specific Gravity and water absorption

- The specific gravity of a solid is the ratio of mass of solid to that of an equal volume of distilled water at a specified temperature.
- Because the aggregates may contain water-permeable voids, so two measures of specific gravity of aggregates are used: apparent specific gravity and bulk specific gravity.

Apparent Specific Gravity (G_{app}) : It is computed on the basis of the net volume of aggregates i.e. the volume excluding water-permeable voids. Thus

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

where, M_D is the dry mass of the aggregate, V_N is the net volume of the aggregates excluding the volume of the absorbed matter, W is the density of water.

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

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

where, V_B is the total volume of the aggregates including the volume of absorbed water.

Water absorption :

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

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

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

Bitumen adhesion test

- Bitumen adheres well to all normal types of road aggregates provided they are dry and free from dust.
- In the absence of water, there is practically no adhesion problem of bituminous construction. Adhesion problem occurs when the aggregate is wet and cold.
- This problem can be dealt with by removing moisture from the aggregate by drying and increasing the mixing temperature.
- Further, the presence of water causes stripping of binder from the coated aggregates. This problem occurs when bitumen mixture is permeable to water.

- Several laboratory tests are conducted to arbitrarily determine the adhesion of bitumen binder to an aggregate in the presence of water.
- Static immersion test is one specified by IRC and is quite simple. The principle of the test is by immersing aggregate fully coated with binder in water maintained at 40°C temperature for 24 hours.

Note: IRC has specified maximum stripping value of aggregates should not exceed 5%.

Property of aggregate	Type of Test
Crushing strength	Crushing test
Hardness	Los Angeles abrasion test
Toughness	Aggregate impact test
Durability	Soundness test-accelerated durability test
Shape factors	Shape test
Specific gravity and porosity	Specific gravity test and water absorption test
Adhesion to bitumen	Stripping value of aggregate

BITUMEN

Bituminous materials are widely used in road construction and maintenance. After gaining experience from their use in obtaining smooth riding surface, bituminous mixtures are being used as structural layers. These materials are considered to be flexible from structural point of view.

Source

The most common source of bitumen is through petroleum crude. It is also found as rock asphalt in some parts of Europe and as Lake Asphalt in Trinidad. Indian crude does not yield good bitumen suitable for roadwork, except Digboi bitumen (in Assam). Thus, India gets its entire bitumen through imported crude.

DESIRABLE PROPERTIES OF BITUMEN

In road construction one looks for the following desirable properties in bitumen.

(a) Bitumen should have good affinity to aggregates.

(b) Bitumen should be fluid enough to coat all particles of the aggregates in a premix process. This is achieved by heating the bitumen and the aggregates too.

(c) In spraying work, bitumen should be susceptible of being sprayed to a thin film. This is achieved either by heating it or by fluxing it or by emulsifying it.

(d) When the binder cools down to atmospheric temperature it should harden to hold the aggregates together.

(e) Its susceptibility to change its viscosity when temperature varies should be low. In particular, its viscosity characteristics should be reasonably constant within the range of temperatures the road experiences.

(f) The bitumen should retain its properties over a long period. In other words, it should be a durable binder and should not lose its properties too soon.

Physical Properties of Bitumen

Bitumen possesses the following properties:

(a) It is a viscous liquid; black or brown in colour
(b) It consists predominantly of hydrocarbons derived from petroleum crude
(c) It is soluble in carbon disulphide
(d) It is insoluble in water
(e) Its specific gravity is around 1.00
(f) It has water-proofing properties
(g) It is thermoplastic, i.e. it becomes soft on heating and in the reverse process becomes hard on cooling.
(h) It oxidies slowly.
(i) It is chemically inert.

TESTING OF BITUMEN

Viscosity

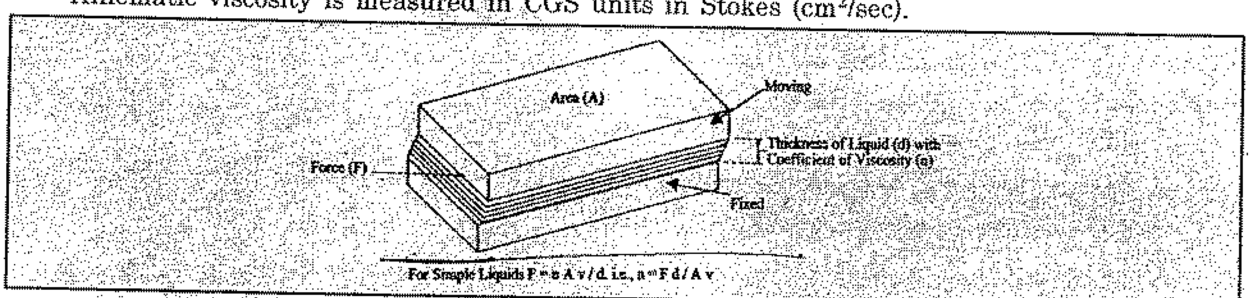
- Viscosity is the property of bitumen which resists flow due to internal friction.
- Absolute or dynamic viscosity is measured by means of a sliding plate viscometer (see figure). A thin film of bitumen of thickness d(20 to 50 microns) is held between two plates, the area of contact being A. The tangential force F to move the top plate at a velocity V is measured. The coefficient of viscosity, η is then given by:

$$\eta = \frac{F.d}{A.V}$$

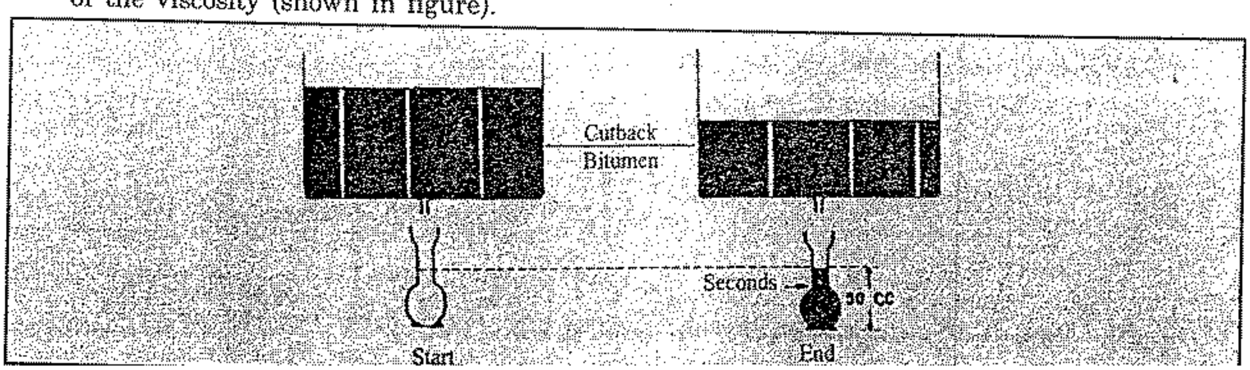
In CGS units it is measured in Poise.

$$\text{Kinematic Viscosity} = \frac{\text{Dynamic viscosity}}{\text{Density}} = \frac{\eta}{\rho}$$

Kinematic viscosity is measured in CGS units in Stokes (cm^2/sec).

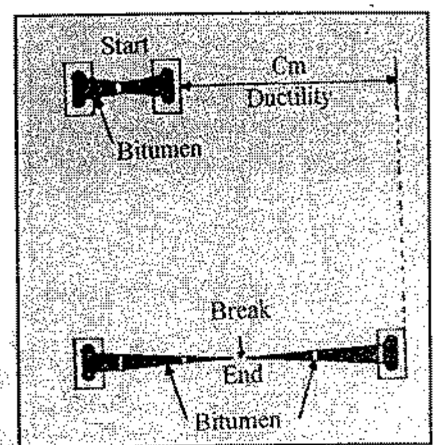


- The viscosity of liquid bitumen is measured by efflux viscometers.
- The liquid is kept at a constant temperature and is made to pass through an orifice; the time required to pass a measured quantity through the orifice is noted and is an indirect measure of the viscosity (shown in figure).



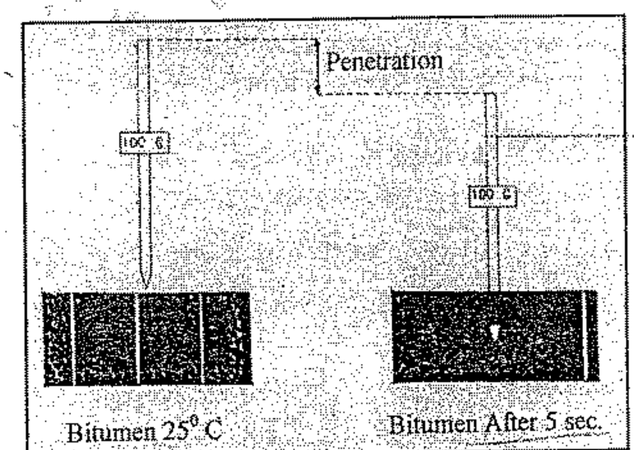
- The methods used are STV (Standard Tar Viscometer), Saybolt Furol, Redwood and Engler.
- The diameter of orifice varies in the range of 3 to 10 mm and the volume of fluid is in the range of 50 to 200 ml.
- Furol Viscosity is standardized test.

Ductility



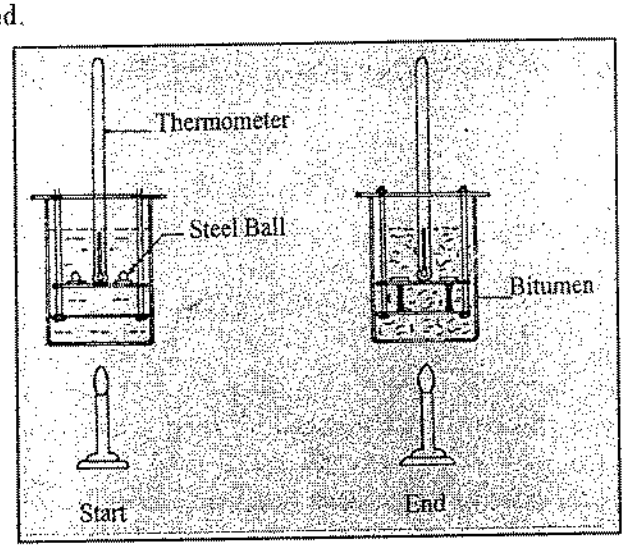
- Bitumen binder should be sufficiently ductile, i.e, it should be capable of being stretched without breaking. Ductility is the opposite of brittleness.
- Ductility is measured by stretching a standard briquette of bitumen (in figure) having a cross-sectional area of 1 sq cm at a temp.of 27°C the rate of pull being 5 cm/min.
- The distance in cm that the briquette can be stretched before breaking is the ductility.
- Its value varies from 5 to 100
- A minimum value of 50 is commonly specified. For waxy bitumen, values may be as low as 15.
- ISI has recommended a minimum ductility value of 75 cm for grades of 45 and above.

Penetration



- A measure of the hardness of bitumen is indirectly obtained by the penetration test.
- It measures the distance a standard blunt-pointed needle will vertically penetrate a sample of material at 27°C, the load being 100 g and time of application of load being 5 secs (in figure).
- The unit of penetration is 1/10 mm.
- Thus 80/100 pen means a penetration of 8-10 mm
- Penetration limits enable bitumen to be classified on the basis of consistency. Common grades are 30/40, 60/70 and 80/100.
- Tars are soft and penetration test is not used.

Softening Point

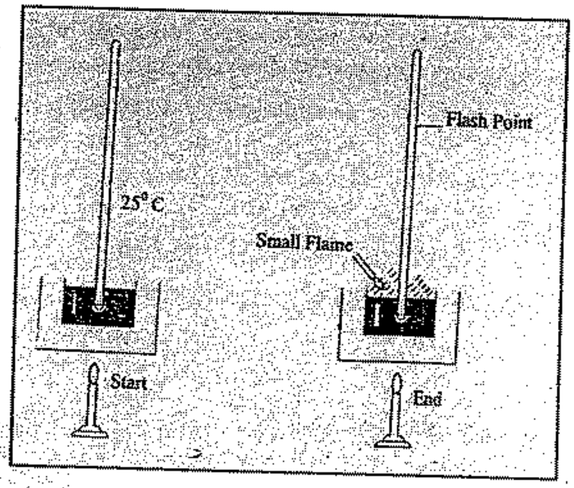


- The softening point is measured by the "*Ring and Ball*" test.
- Softening point is the temperature at which bitumen attains a particular degree of softness under standardised testconditions.
- The temperature at which a standard steel ball placed on a layer of bitumen kept in a standard ring passes through the bitumen layer and touches the bottom plate kept at a distance of 2.54 cm is the softening point.

- The both liquid is water/glycerine. The softening point of paving grade bitumen 80/100 is 35–50°C. The significance of the softening point is that it indicates the temperature at which bitumen passes from solid to liquid consistency.
- For satisfactory performance and avoidance of "bleeding", bitumen should have a softening point 5 to 10°C above the maximum atmospheric temperature.

Flash and Fire Point

- At high temperature, bitumen becomes volatile, and thus catches fire which is very hazardous. Thus it is necessary that we quantify this temperature for each grade of bitumen.
- The flash point (shown in figure) is the lowest temperature in degrees C at which the application of a test flame causes the vapour from bitumen to catch fire momentarily in the form of a flash.
- The fire point is the lowest temperature in degrees C at which the application of the test flame causes the bitumen to ignite and burn for at least 5 secs under specified conditions of test.
- The safe limit for heating bitumen is normally 50°C below the flash point.



Specific Gravity

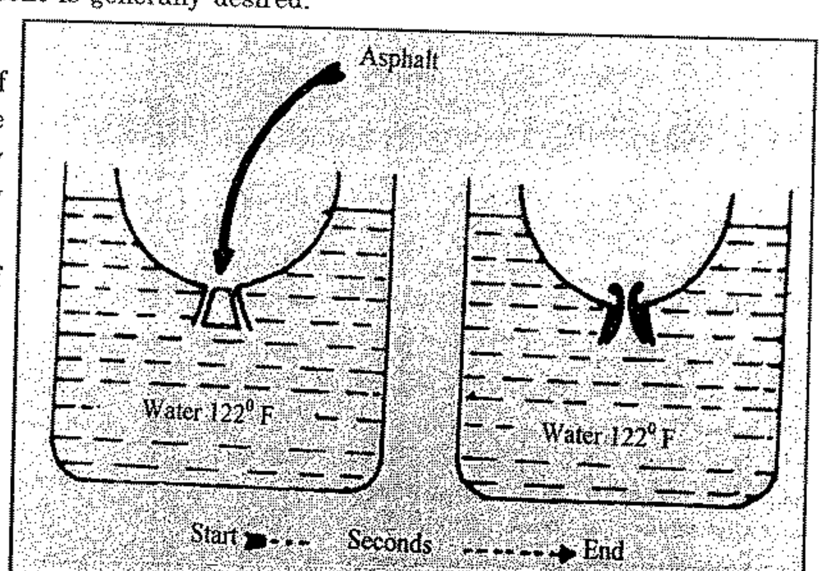
- Specific gravity of bitumen is determined by the pycnometer method.
- The measurements are taken at 27°C. The specific gravity is generally around 1.00.
- Specific gravity of pure bitumen is in the range of 0.97 to 1.02 where as Tars have specific gravity ranging from 1.10 to 1.25.

Solubility

- The solubility of bitumen in trichloroethylene is a measure of its purity.
- A minimum value of 99 percent is generally desired.

Float Test

- Normally the consistency of bituminous material can be measured either by penetration test or by viscosity test.
- But for certain range of consistencies, these tests are not applicable and float test is used.
- The float test is a modified viscosity test adopted for use with small quantities of very viscous bituminous materials.



- It is the time in seconds required for a small plug of chilled bitumen, which is held in an open mould attached to the

bottom of a saucer, to become sufficiently fluid when the saucer is floated in water at 50°C.

Loss on Heating

- When a bituminous material is heated, it loses its volatile and therefore hardness.
- The less the loss on heating, the better is the bitumen.
- The test is conducted by an accelerated heat test. 50 g of bitumen is placed in a container in one of the recesses of revolving shelf. The shelf rotates at 5 to 6 revolutions per minute. The test is carried out for 5 hrs in an oven at 163°C.
- Not more than one percent loss in weight is desirable.

Water Content Test

It is desirable that the bitumen contains minimum water content to prevent foaming of bitumen when it is heated above the boiling point of water. The maximum water content in bitumen should not exceed 0.2% by weight.

SELECTION OF APPROPRIATE GRADE OF BITUMEN

Guidelines for selection of bitumen grade in India are given below:

Type of Bitumen	Use
Penetration Grade 30/40	Hot-mix work in areas where the difference between maximum and minimum temperature is less than 25°C and on roads with high volume of traffic (expressways, urban roads and factory roads)
Penetration Grade 60/70	Hot-mix work for bituminous macadams and bituminous concrete for superior type of roads with high traffic and in normal summer temperatures. (i) For surface dressing. (ii) For premix works in high altitudes. (iii) For premix works in roads with less traffic intensity.
Cutback Bitumen	(i) Surface dressing in cold weather (ii) Premix in cold weather (iii) MC and SC cut backs are used for priming.
Emulsion	(i) Used for surface dressing in cold weather, wet conditions and maintenance works (ii) Used for premix works in wet weather (except dense and semi-dense carpet and maintenance works. (iii) Priming.

Cutback Bitumen

The viscosity of bitumen is reduced by a volatile diluents. Cut back bitumens are available in three types.

1. Rapid Curing (RC) 2. Medium Curing (MC) 3. Slow Curing (SC)

- The cutbacks are designated by numerals representing progressively thicker or viscous cutback. For example RC-2 is more thick than RC-1 but RC-2, MC-2 and SC-2 have same viscosity.
- RC-0 and SC-0 may have 45% solvent and 55% bitumen where as RC-5 and MC-5 may contain 15% solvent and 85% bitumen.
- *RC-Cutback:* They have penetration value of 80 to 120 *eg.*, petroleum such as naptha or gasoline,
- *MC-Cutback:* They have good wetting properties. *eg.*, kerosene and light diesel oil.
- *SC-cutback:* These can be obtained by blending bitumen with high boiling point gas oil or by controlling the rate of flow and temperature of crude during the first cycle of refining.

Bituminous Emulsion

- Emulsion is a two phase system consisting of two immiscible liquids.
- The bitumen/tar content in emulsion range from 40 to 60% and the remaining portion is water.
- The average diameter of globules of bitumen portion is about 2 mm.
- Emulsion are used especially in maintenance and patch repair works. The main advantage of emulsion is that is can be used in wet weather even when it is raining. Emulsions can be used for soil stabilization in deserts.

Tar

Tar can be produced in 3 stages

(i) Carbonization of coal to produce crude tar.

(ii) Refining or distillation of crude tar.

(iii) Blending of distillation residue with distillate oil fraction to give desired road tar.

- RT-1 is lowest viscosity used for surface painting where as RT-4 may be used for premix in macadam.
- RT-5 is used for grouting which has highest viscosity.

Tar and Bitumen (Comparison)

- Bitumen is a petroleum product whereas tar is produced by the destructive distillation of coal or wood.
- Bitumen is soluble in 'carbon disulphide' and 'carbon tetrachloride' but tar is soluble in only toluene.
- Bitumen is more resistant to water than tar.
- Tar is more temperature susceptible resulting in great variation in viscosity with temperature.
- The free carbon content is more in tar as seen from the solubility test.

MIX DESIGN METHODS

There are four popular methods of mix design:

1. Marshall method
2. Hubbard-Field method
3. Hveem method
4. Smith traxial method

- Each of the above methods is associated with a set of design criteria for the properties of the mix.
- The Marshall method is the most popular in India.

Objectives of mix design

The objective of the mix design is to produce a bituminous mix by proportioning various components so as to have.

1. sufficient bitumen to ensure a durable pavement,
2. sufficient strength to resist shear deformation under traffic at higher temperature,
3. sufficient air voids in the compacted bitumen to allow for additional compaction by traffic,
4. sufficient workability to permit easy placement without segregation,
5. sufficient flexibility to avoid premature cracking due to repeated bending by traffic, and
6. sufficient flexibility at low temperature to prevent shrinkage cracks.

Constituents of a mix

(1) **Coarse aggregates :** Offer compressive and shear strength and shows good interlocking properties. *e.g.*, Granite

(2) **Fine aggregates :** Fills the voids in the coarse aggregate and stiffens the binder. *e.g.*, Sand, Rock dust

(3) **Filler :** Fills the voids, stiffens the binder and offers permeability. *e.g.*, Rock dust, cement, lime

(4) **Binder :** Fills the voids, cause particle adhesion and gluing and offers impermeability. *e.g.*, Bitumen, Asphalt, Tar

Types of mix

(1) **Well-graded mix :** Dense mix, bituminous concrete has good proportion of all constituents and are called dense bituminous macadam, offers good compressive strength and some tensile strength.

(2) **Gap-graded mix :** Some large coarse aggregates are missing and has good fatigue and tensile strength.

(3) **Open-graded mix :** Fine aggregate and filler are missing, it is porous and offers good friction, low strength and for high speed.

(4) **Unbounded :** Binder is absent and behaves under loads as if its components were not linked together, though good interlocking exists. Very low tensile strength and needs kerb protection.

Different layers in a pavement

(1) **Bituminous base course :** Consist of mineral aggregate such as stone, gravel, or sand bonded together by a bituminous material and used as a foundation upon which to place a binder or surface course.

(2) **Bituminous binder course :** A bituminous-aggregate mixture used as an intermediate course between the base and surface courses or as the first bituminous layer in a two-layer bituminous resurfacing. It is sometimes called a levelling course.

(3) **Asphaltic/Bituminous concrete :** Bituminous concrete consists of a mixture of aggregates continuously graded from maximum size, typically less than 25 mm, through fine filler that is smaller than 0.075 mm. Sufficient bitumen is added to the mix so that the compacted mix is effectively impervious and will have acceptable dissipative and elastic properties.

REQUIREMENT OF BITUMINOUS MIXES

Stability

- Stability is defined as the resistance of the paving mix to deformation under traffic load.
- Two examples of failure are (i) *shoving* - a transverse rigid deformation which occurs at areas subject to severe acceleration and (ii) *grooving* - longitudinal ridging due to channelization of traffic.
- Stability depend on the inter-particle friction, primarily of the aggregates and the cohesion offered by the bitumen.
- Sufficient binder must be available to coat all the particles, at the same time should offer enough liquid friction.
- However, the stability decreases when the binder content is high and when the particles are kept apart.

Durability:

- Durability is defined as the resistance of the mix against weathering and abrasive actions.
- Weathering causes heardening due to loss of volatiles in the bitumen.
- Abrasion is due to wheel loads which causes tensile strains.
- Typical examples of failure are (i) *pot-holes* - deterioration of pavements locally and (ii) *stripping* lost of binder from the aggregates and aggregates are exposed.
- Distingration is minimized by high binder content since they cause the mix to be air and waterproof and the bitumen film is more resitant to hardening.

Flexibility:

- Flexibility is a measure of the level of bending strength needed to counteract traffic load and prevent cracking of surface.
- Fracture is the cracks formed on the surface (hair-line-cracks, alligator cracks), main reasons are shrinkage and brittleness of the binder.
- Shrinkage cracks are due to volume change in the binder due to aging.
- Brittleness is due to repeated bending of the surface due to traffic loads.
- Higher bitumen content will give better flexibility and less fracture.

Skid Resistance :

- It is resistance of the finished pavement against skidding which depends on the surface texture and bitumen content.
- It is an important factor in high speed traffic.
- Normally, an open gradedd coarse surface texture is desirable.

Workability :

- Workability is the ease with which the mix can be laid and compacted, and formed to the requried condition and shape.

- This depends on the gradation of aggregates,their shape and texture, bitumen content and its type.
- Angular, flaky, and elongated aggregates reduces workability. On the other hand, rounded aggregates improve workability

Desirable properties

From the above discussion, the desirable properties of a bituminous mix can be summarized as follows:

- Stability to meet traffic demand
- Bitumen content to ensure proper binding and water proofing
- Voids to accommodate compaction due to traffic
- Flexibility to meet traffic loads, esp. in cold season
- Sufficient workability for construction
- Economical mix

DRY MIX DESIGN

- The objective of dry mix design is to determine the amount of various sizes of mineral aggregates to use to get a mix of maximum density.
- The dry mix design involves three important steps, viz. (i) Selection of aggregates, (ii) Aggregates gradation, and (iii) proportion of aggregates.

Selection of aggregates

- The desirable qualities of a bituminous paving mixture are dependent to a considerable degree on the nature of the aggregates used.
- Aggregates are classified as coarse, fine, and filler.
- The function of the coarse aggregates in contributing to the stability of a bituminous paving mixture is largely due to interlocking and frictional resistance of adjacent particles.
- Similarly, fines or sand contributes to stability failure function in filling the voids between coarse aggregates.
- Mineral filler is largely visualized as a void filling agent. Crushed aggregates and sharp sands produce higher stability of the mix when compared with gravel and rounded sands.

Proportioning of aggregates

After selecting the aggregates and their gradation, proportioning of aggregates has to be done and following are the common methods of proportioning of aggregates:

(i) Trial and error procedure

(ii) Graphical Methods

(iii) Analytical Method

MARSHALL MIX DESIGN

- The mix design (wet mix) determines the optimum bitumen content.

For evaluation of performance of Bitumenous mixes, flow test & stability test are performed.

- The Marshall stability and flow test provides the performance prediction measure for the Marshall mix design method.
- The stability portion of the test measures the maximum load supported by the test specimen at a loading rate of 50.8 mm/minute.
- Load is applied to the specimen till failure, and the maximum load is designated as stability.
- Stability is defined as maximum load carried by specimen at a standard temperature of 60°C. It is expressed in Kg.
- Flow is measured as deformation in units of 0.25 mm between as load & maximum load during stability test. Thus if deformation is 4mm, flow value is 16.

Specimen preparation

- Approximately 1200 gm of aggregates and filler is heated to a temperature of 175–190°C.
- Bitumen is heated to a temperature of 121–125°C with the first trial percentage of bitumen (say 3.5 or 4% by weight of the mineral aggregates).
- The heated aggregates and bitumen are thoroughly mixed at a temperature of 154–160°C.
- The mix is placed in a preheated mould and compacted by a rammer with 50 blows on either side at temperature of 138°C to 149°C.
- The weight of mixed aggregates taken for the preparation of the specimen may be suitably altered to obtain a compacted thickness of 63.5 ± 3 mm.
- Vary the bitumen content in the next trial by +0.5% and repeat the above procedure upto about 8% bitumenous.

Properties of the mix

The properties that are of interest include the

(1) Theoretical specific gravity (G_t)

(2) The bulk specific gravity of the mix (G_m)

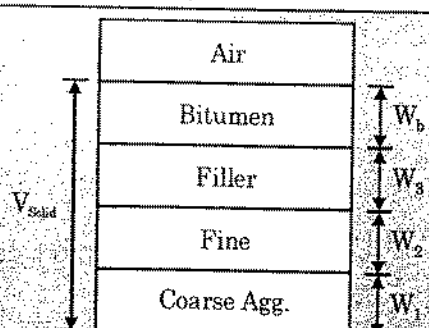
(3) percent air voids (V_v)

(4) percent volume of bitumen (V_b)

(5) percent void in mixed aggregate (VMA) and percent.

1. Theoretical specific gravity of the mix (G_t)

Theoretical specific gravity (G_t) is the specific gravity without considering air voids, and is given by



$$G_t = \frac{W_{total}}{V_{solid}\,\gamma_w}$$

$$G_t = \frac{W_1 + W_2 + W_3 + W_b}{\frac{W_1}{G_1} + \frac{W_2}{G_2} + \frac{W_3}{G_3} + \frac{W_b}{G_b}} = \frac{\text{Total weight}}{(\text{Volume of solids})\gamma_w} \quad \ldots (i)$$

where, W_1 is the weight of the coarse aggregate in the total mix, W_2 is the weight of fine aggregate in the total mix, W_3 is the weight of filler in the total mix, W_b is the weight of bitumen in the total mix, G_1 is the apparent specific gravity of coarse aggregate, G_2 is the apparent specific gravity of fine aggregate, G_3 is the apparent specific gravity of filler and G_b is the apparent specific gravity of bitumen.

2. Bulk specific gravity of mix (G_m)

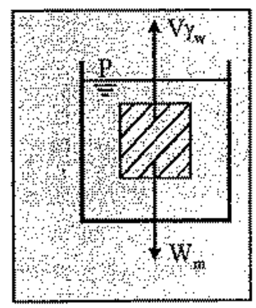
The bulk specific gravity or the actual specific gravity of the mix (G_m) is the specific gravity considering air voids and is found out by:

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

$$G_m = \frac{W_{total}}{(V_{solid} + V_{air})\gamma_w} \quad \text{... (ii)}$$

$$G_t > G_m \text{ always}$$

Where, W_m is the weight of mix in air (actual wt), W_w is the weight of mix in water (Bouyant weight).



$$W_m - V\gamma_w = W_w = \text{Bouyant weight}$$

$$\Rightarrow \quad V = \frac{W_m - W_w}{\gamma_w}$$

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

3. Air voids percent (V_v)

Air voids (V_v) is the percent of air voids by volume in the specimen and is given by:

$$V_v = \frac{(G_t - G_m)100}{G_t} = V_v = \frac{V_{void(air)}}{V_{total}} \times 100 \quad \text{... (iii)}$$

This can be proved as follows,

$$\frac{G_t - G_m}{G_t} \times 100 = 100 \times \frac{\dfrac{W_{total}}{V_{solid}\gamma_w} - \dfrac{W_{total}}{(V_{solid} + V_{air})\gamma_w}}{\dfrac{W_{total}}{V_{solid}\gamma_w}}$$

$$= 100\left[1 - \frac{V_{solid}}{V_{solid} + V_{air}}\right] = \frac{V_{air} \times 100}{V_{solid} + V_{air}}$$

4. Percent volume of bitumen (V_b)

The percent volume of bitumen (V_b) is the volume of bitumen to the total volume and is given by:

$$V_b = \frac{\dfrac{W_b}{G_b}}{\dfrac{W_1 + W_2 + W_3 + W_b}{G_m}} \times 100 = V_b = \frac{V_{bitumin}}{V_{total}} \times 100 \quad \text{... (iv)}$$

where, W_1 is the weight of coarse aggregate in the total mix, W_2 is the weight of fine aggregate in the total mix, W_3 is the weight of filler in the total mix, W_b is the weight of bitumen in the total mix, G_b is the apparent specific gravity of bitumen, and G_m is the bulk specific gravity of mix given by equation (ii).

5. Voids in mineral aggregate (VMA)

Voids in mineral aggregate (VMA) is the volume of voids in the aggregates and is the sum of air voids and volume of bitumen and is calculated from

$$VMA = V_v + V_b \quad ...(v)$$

where, V_v is the percent air voids in the mix, given by equation (iii) and V_b is percent bitumen content in the mix, given by equation (iv)

$$VMA = \frac{V_{air}}{V_{Total}} + \frac{V_{bitumen}}{V_{Total}} = \frac{V_{air} + V_{bitumen}}{V_{Total}}$$

6. Voids filled with bitumen (VFB)

Voids filled with bitumen (VFB) is the voids in the mineral aggregate frame work filled with the bitumen, and is calculated as:

$$VFB = \frac{V_b \times 100}{VMA} = VFB = \frac{\frac{V_{bitumen}}{V_{Total}} \times 100}{\frac{V_{air}}{V_{Total}} + \frac{V_{bitumen}}{V_{Total}}} = \frac{V_{bitumen}}{V_{air} + V_{bitumen}} \quad ...(vi)$$

where, V_b is percent bitumen content in the mix, given by equation (iv) and VMA is the percent voids in the mineral aggregate, given by equation (v).

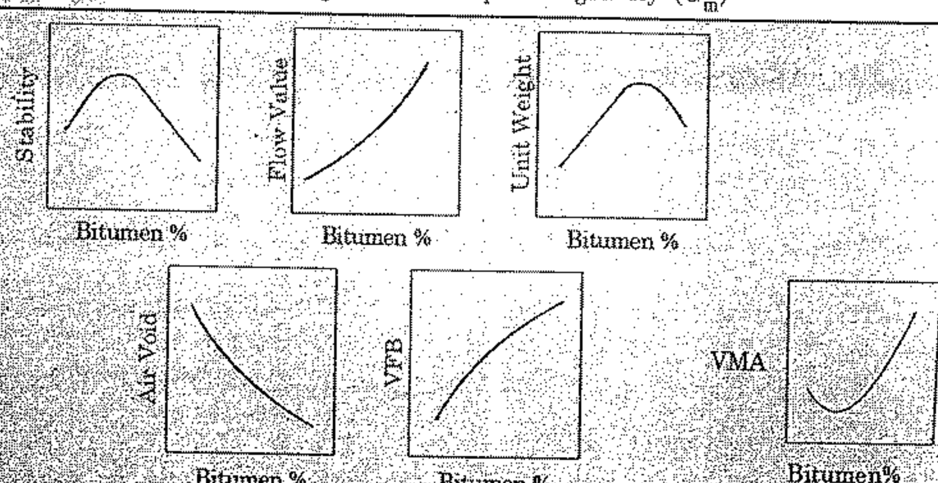
Determine Marsball stability and flow

- Marshall stability of a test specimen is the maximum load required to produce failure when the specimen is preheated to a prescribed temperature placed in a special test head and the load is applied at a constant strain (5 cm/min)
- While the stability test is in progress dial gauge is used to measure the vertical deformation of the specimen. The deformation at the failure point expressed in units of 0.25 mm is called the Marshall flow value of the specimen.

Prepare graphical plots

The average of the above properties are determined for each mix with different bitumen content and the following graphical plots are prepared:

1. Binder content versus corrected Marshall stability.
2. Binder content versus Marshall flow
3. Binder content versus percentage of void (V_v) in the total mix.
4. Binder content versus voids filled with bitumen (VFB)
5. Binder content versus unit weight or bulk specific gravity (G_m)



Marshall graphical plots

Determine optimum bitumen content

- Determine the optimum binder content for the mix design by taking average value of the following three bitumen contents found from the graphs obtained in the previous step.
  1. Binder content corresponding to maximum stability
  2. Binder content corresponding to maximum bulk specific gravity (G_m)
  3. Binder content corresponding to the designed limits of percent air voids (V_v) in the total mix (i.e. 4%)
- The stability value, flow value, and VFB are checked with Marshall mix design specification chart given in Table below.
- Mixes with very high stability value and low flow value are not desirable as the pavements constructed with such mixes are likely to develop cracks due to heavy moving loads.

Marshall Mix design specification

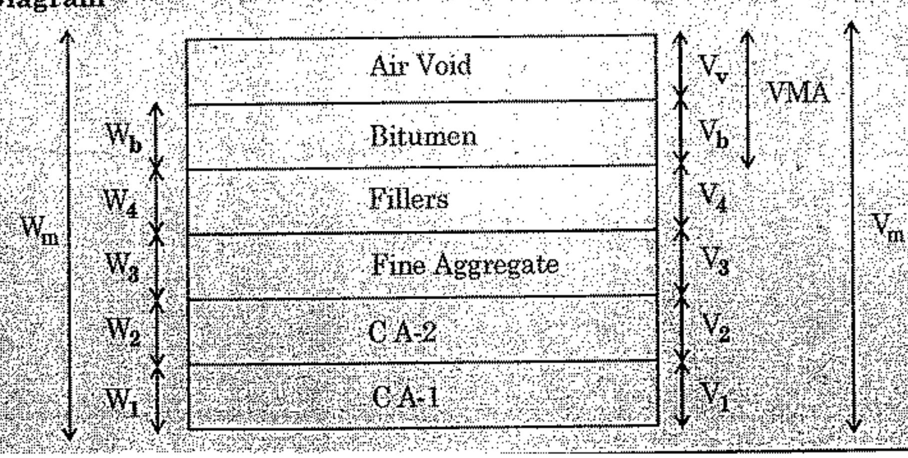
Test Property	Specified Value
Marshall stability, kg	340 (minimum)
Flow value, 0.25 mm units	8 – 16
Percent air voids in the mix V_v%	3 – 5
Voids filled with bitumen VFB%	75 – 85

Example 1

The specific gravities and weight proportion for aggregrate and bitumen are as under for the preparation of Marshall mix design. The volume and weight of one Marshall specimen was found to be 475 cc and 1100 gm. Assuming absorption of bitumen in aggregrate is zero. Find V_v, V_b, VMA and VFB

Item	CA1	CA2	FA	Filler	Bitumen
Wt (gm)	825	1200	325	150	100
Sp. Gr	2.63	2.51	2.46	2.43	1.05

Sol. Phase Diagram



Given Data : $W_1 = 825$; $W_2 = 1200$; $W_3 = 325$; $W_4 = 150$; $W_b = 100$

$G_1 = 2.63$; $G_2 = 2.51$; $G_3 = 2.46$; $G_4 = 2.43$; $G_S = 1.05$

$$\therefore \quad G_t = \frac{(W_1 + W_2 + W_3 + W_4 + W_b)}{\left(\frac{W_1}{G_1} + \frac{W_2}{G_2} + \frac{W_3}{G_3} + \frac{W_4}{G_4} + \frac{W_b}{G_b}\right)} = \frac{(825 + 1200 + 325 + 150 + 100)}{\left(\frac{825}{2.63} + \frac{1200}{2.51} + \frac{325}{2.46} + \frac{150}{2.43} + \frac{100}{1.05}\right)} = \frac{2600}{1080.86} = 2.406$$

$$G_m = \frac{1100\text{g}}{475\text{cc} \times 1\text{g/cc}} = 2.316$$

$$\text{Percent of Air Void, } V_v = \left(\frac{G_t - G_m}{G_t}\right) \times 100 = \left(\frac{2.406 - 2.316}{2.406}\right) \times 100 = 3.74\%$$

$$\text{Percent volume of bitumen, } V_b = \frac{\left(\frac{W_b}{G_b}\right) \times 100}{\left(\frac{W_1 + W_2 + W_3 + W_4 + W_b}{G_m}\right)} = \frac{\frac{100}{1.05}}{\frac{(825 + 1200 + 325 + 150 + 100)}{2.316}} \times 100 = 8.48\%$$

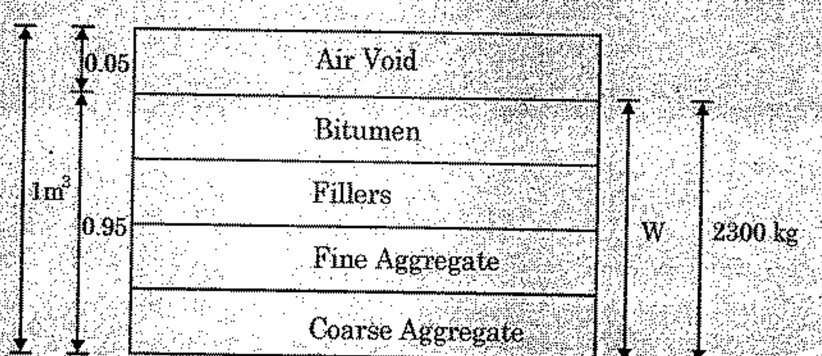
$$\therefore \quad \text{VMA} = (V_v + V_b) = (3.74 + 8.48) = 12.22\%$$

$$\text{VFB} = \frac{V_b \times 100}{\text{VMA}} = \frac{8.48 \times 100}{12.22} = 69.39\%$$

Example 2

A mixture contains coarse aggregrate (S.G = 2.7) fine aggregate (S.G = 2.9) and Mineral Filler (S.G = 1.5) in proper ratio 60 : 35 : 5 by weight. These materials when mixed with bitumen (S.G = 1.01) and compacted to a unit weight of 2300 kg/m³ contains 5% voids. How much bitumen does the specimen contain? How will you arrive at the optimum bitumen content based on the curves plotted in Marshall's Test.

Sol. Given Data : $G_1 = 2.7$ $G_2 = 2.9$ $G_3 = 1.5$



$$\text{C.A} = \frac{60W}{100} = 0.60W \text{ ; F.A} = \frac{35W}{100} = 0.35W \text{ ; M.F} = \left(\frac{5W}{100}\right) = 0.05W$$

Let the weight of bitumen be W_b kg.

$$\therefore \quad W_b + 0.60W + 0.35W + 0.05W = 2300$$

$$W = (2300 - W_b) \quad \text{... (i)}$$

From Phase Diagram

$$\frac{0.60W}{2.7\rho_w} + \frac{0.35W}{2.9\rho_w} + \frac{0.05W}{1.5\rho_w} + \frac{W_b}{1.01\rho_w} = 0.95$$

$$\Rightarrow \quad \frac{W}{\rho_w}\left[\frac{0.60}{2.7} + \frac{0.35}{2.9} + \frac{0.05}{1.5}\right] + \frac{W_b}{1.01\rho_w} = 0.95$$

$$\Rightarrow \quad \frac{0.376W}{\rho_w} + \frac{W_b}{1.01\rho_w} = 0.95 \quad \text{... (ii)}$$

Put the value of (i) in (ii)

$$\Rightarrow \quad 0.376(2300 - W_b) + \left(\frac{W_b}{1.01}\right) = 0.95\rho_w$$

$$\Rightarrow \quad W_b\left[\frac{1}{1.01} - 0.376\right] = (0.95 \times 1000 - 0.376 \times 2300)$$

$$W_b = \frac{(0.95 \times 1000 - 0.376 \times 2300)}{\left[\frac{1}{1.01} - 0.376\right]} = 138.74 \text{ kg}$$

$$W = (2300 - 138.74) = 2161.26 \text{ kg}$$

$$\text{C.A.} = (0.60 \times 2161.26) = 1296.76 \text{ kg.}$$

$$\text{F.A.} = (0.35 \times 2161.26) = 756.44\text{kg}$$

$$\text{M.F.} = (0.05 \times 2161.26) = 108.063 \text{ kg}$$

Five curve are plotted with bitumen content on X-axis and the following values on the y-axis

(i) Marshall stability value (ii) Flow value

(iii) Unit Weight (iv) % voids in total mixture

(v) % voids filled with bitumen

The optimum bitumen content for the mixture design is found by taking the average value of the following contents from the graphs of the test results

(i) Bitumen content c/p to maximum stability

(ii) Bitumen content c/p to maximum unit weight

(iii) Bitumen content c/p to the median of designed limits of % air voids in the total mixture(4%)

Example 3

A specimen of bituminous concrete has a height of 6.5 cm and a diameter of 10 cm. The weight of the compacted specimen in air is 1,230g and its weight when immersed in water is 665g. When coated with paraffin its weight is 1,260g and its weight when immersed in water is 661 g. Specific gravity of paraffin is 0.90. The analysis of the specimen yielded the following data:

S.No.	Material	Specific Gravity (g/cc)	Mix Composition (% by Weight of Total Mix)	Aggregate Composition (% by Weight of Total Aggregates)
1	Bitumen	1.02	5.0	
2	Coarse aggregates	2.60	53.0	56.3
3	Fine aggregates	2.65	36.0	38.0
4	Filler	2.68	6.0	5.7

Calculate the following.

(a) Bulk density of uncoated specimen by the immersion procedure
(b) Bulk density of specimen from specimen dimensions
(c) Bulk density of specimen by paraffin coating procedure.
(d) Average specific gravity of aggregates
(e) Maximum theoretical density.
(f) Bulk density as percent of maximum density
(g) Percent voids in compacted mix.
(h) Percent of volume occupied by bitumen, coarse aggregates and filler.
(i) Percent volume of voids in mineral aggregate
(j) Percent voids in aggregates filled with bitumen.

Sol. (a) Bulk density of uncoated specimen by immersion procedure.

$$= \text{Weight in air / Volume}$$

$$= \text{Weight in air / Loss of weight when immersed in water}$$

$$= \frac{1230}{1230-665} = \frac{1230}{565} = 2.18 \text{ g/cc}$$

(b) Bulk density from specimen dimensions

$$= \text{Weight / Volume} = \frac{1230}{\frac{3.142\times10\times10\times6.5}{4}} = 2.41\text{g/cc}$$

(c) Bulk density by paraffin-coating procedure

$$= \frac{\text{Wt. of uncoated specimen in air}}{\text{Wt. of paraffin coated specimen in air} - \text{Wt. of paraffin coated specimen in water} - \frac{[(\text{Wt. of coated specimen}) - (\text{Wt. of uncoated specimen})]}{\text{Specific gravity of paraffin}}}$$

$$= \frac{1230}{1260-661-\frac{1260-1230}{0.90}} = \frac{1230}{1260-661-33.3} = \frac{1230}{565.7} = 2.17\text{g/cc}$$

(d) Avg. specific gravity of aggregates $= \dfrac{100}{\frac{56.3}{2.6}+\frac{38}{2.65}+\frac{5.7}{2.68}} = \dfrac{100}{38.12} = 2.62$ g/cc

(e) Maximum theoretical density $= \dfrac{\text{Weight in air}}{\text{Vol. of bitumen + Vol. of aggregates}}$

$$= \frac{1230}{\dfrac{0.050 \times 1230}{1.02} + \dfrac{0.950 \times 1230}{2.62}} = \frac{1230}{506.29} = 2.43$$

(f) Bulk density as percent of maximum theoretical density $= \dfrac{2.18}{2.43} = 89.7\%$

(g) Percent voids in compacted mix = (Total volume − Volume of ingredients × 100)

$$= \frac{\dfrac{1230}{2.18} - \dfrac{1230 \times 0.05}{1.02} - \dfrac{1230 \times 0.53}{2.60} - \dfrac{1230 \times 0.36}{2.65} - \dfrac{1230 \times 0.06}{2.69}}{\dfrac{1230}{2.18}}$$

$$= 0.1083 = 10.38\%$$

(h) Percent of volume occupied by bitumen, coarse aggregates, fine aggregates and filler.

% volume occupied by bitumen = (5.00 × 2.18) / 1.02 = 10.69

% volume occupied by coarse aggregates = (53.0 × 2.18)/2.60 = 44.44

% volume occupied by fine aggregates = (36.0 × 2.18)/2.68 = 29.62

% volume occupied by filler = (6.0 × 2.18)/2.68 = 4.88

(i) Percent volume of voids in mineral aggregates

$$= \frac{(100 - \%\text{wt. of aggregates} \times 1230 \times 100)}{[\text{Avg. specific gravity of aggregates} \times (1230 - 665]}$$

$$= 100 - \frac{0.95 \times 1230}{2.41 \times 565} \times 100 = 100 - 85.82 = 14.18\%$$

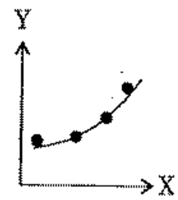
(j) Percent of voids filled with bitumen $= \dfrac{\text{Percent of bitumen in mix by volume} \times 100}{\text{percent of voids in aggregates}}$

$$= \frac{10.69 \times 100}{14.18} = 75.39\%$$

OBJECTIVE QUESTIONS

1. In the Los Angeles Abrasion Test on aggregate, if the speed of the drum is increased to 50 rpm, then the abrasion value will
   (a) increase (b) decrease
   (c) remain unchanged (d) be unpredictable

2. A typical Marshall test graph is shown in the given figure. The variable on the X-axis is % binder content by weight of total mix. The variable on the Y-axis for the given graph will be



   (a) stability value (b) flow value
   (c) percentage of voids (d) unit weight

3. The plasticity index of the fraction passing 425 micron IS sieve in case of sub-base/base course should be
   (a) less than 6 (b) greater than 6
   (c) greater than 9 (d) between 15 and 30

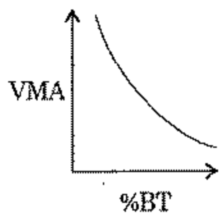
4. Match **List-I** with **List-II** and select the correct answer using the codes given below the lists:

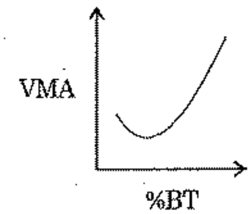
List-I	List-II
A. Disintegration of aggregates due to weathering	**1.** Attrition
B. The removal of material from the surface of the road by grinding action	**2.** Crushing
C. Mutual rubbing or grinding within the mass under the action of traffic	**3.** Soundness
D. Breaking up of road surface layer through cracking into irregular shaped areas	**4.** Abrasion
	5. Disintegration

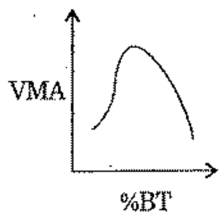
Codes:

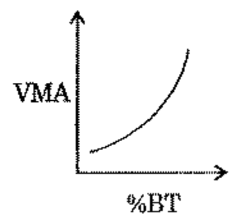
	A	B	C	D
(a)	3	1	5	2
(b)	2	5	1	4
(c)	3	4	1	2
(d)	1	4	3	5

5. Which one of the following diagrams illustrates the relationship between VMA and % bitumen content (BT) in Marshall test?

(a) 

(b) 

(c) 

(d) 

6. If the CBR value obtained at 5 mm penetration is higher than that at 2.5 mm, then the test is repeated for checking; and if the check test reveals a similar trend, then the CBR value is to be reported as the
   (a) mean of the values for 5 mm and 2.5 mm penetration
   (b) higher value minus the lower value
   (c) lower value corresponding to 2.5 mm penetration
   (d) higher value obtained at 5 mm penetration

7. The amount of mechanical energy imposed on the aggregate during the aggregate impact test is of the order of
   (a) 5320 kg-cm (b) 6750 kg-cm
   (c) 7980 kg-cm (d) 11400 kg-cm

8. Which one of the following binders is recommended for a wet and cold climate?
   (a) 80/100 penetration asphalt (b) Tar
   (c) Cutback (d) Emulsion

9. With reference to the Marshall mix design criteria for highways, which one of the following parts is NOT correctly matched?
   (a) Stability value : 340 (minimum)
   (b) Flow value : 8-16
   (c) VFB : 50-75
   (d) % Air voids : 3-5

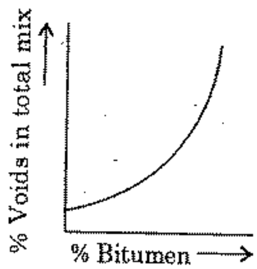
10. Which one of the following pairs is NOT correctly matched?
   (a) Horizontal curves – Superelevation
   (b) O and D studies – Desire lines
   (c) Los Angeles test – Hardness of aggregates
   (d) Soundness test – Purity of bitumen

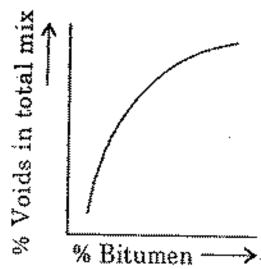
11. **Assertion (A):** In bituminous mixes, the minimum voids requirement provides space for densification under traffic movements and expansion of bitumen at high temperature.
   Reason (R): Insufficient voids in the bituminous mix causes bleeding of the bituminous surface and skidding.

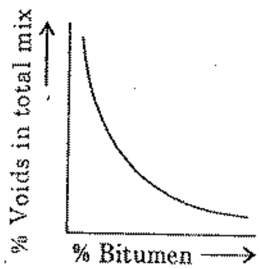
12. If the modulus of subgrade reaction of a soil is 10 kg/cm^3 when tested with a 30 cm diameter plate, the corrected modulus of subgrade reaction for the standard diameter plate will be
   (a) 4 (b) 15
   (c) 20 (d) 25

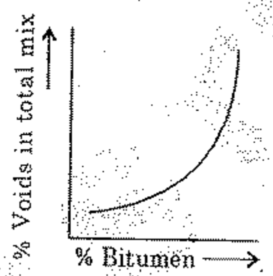
13. Bitumen grade 80/100 indicates that under the standard test conditions, penetration value of bitumen would vary from
    (a) 0.8 mm to 1 mm
    (b) 8 mm to 10 mm
    (c) 8 cm to 10 cm
    (d) 0.08 mm to 0.1 mm

14. Which one of the following curves illustrates the correct relation between % voids in total mix and % bitumen?

(a)


(b)


(c)


(d)


15. Modulus of subgrade reaction using 30 cm diameter plate is obtained as 200 N/cm^3. The value of the same (in N/cm^3) using the standard plate will be
    (a) 500
    (b) 200
    (c) 85
    (d) 80

16. **Assertion (A):** The Marshall test, being a confined compression test has a correlation with deformation as it occurs on the road.

    Reason (R): In road, loaded area is confined by the tyre, the surrounding surfacing and the road base.

17. Match **List-I** (Test) with **List-II** (Properties) and select the correct answer using the codes given below the lists:

List-I	List-II
A. CBR test	**1.** Modulus of sub-grade reaction
B. Plate bearing test	**2.** Arbitrary soil strength
C. Triaxial test	**3.** Exudation and expansion pressure
D. Stabilometer and Cohesion-meter test	**4.** Shear parameters

Codes:

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

18. In 500 gm sample of coarse aggregate, there are 100 gm flaky particles and 80 gm elongated particles. What are the flakiness and elongation indices (total) as per IS?

(a) 40% (b) 36%
(c) 18% (d) 4%

19. California Bearing Ratio (CBR) is a

(a) measure of soil strength
(b) method of soil identification
(c) measure to indicate the relative strengths of paving materials
(d) measure of shear strength under lateral confinement

20. Match **List-I** (Type of binder) with **List-II** (Uses) and select the correct answer using the codes given below the lists:

List-I	**List-II**
A. 80/100 penetration grade bitumen	**1.** Mastic-asphalt
B. 85/25 blown-bitumen	**2.** Bituminous roads
C. MC-70 cutback	**3.** Grouting works
D. RT-5 road tar	**4.** Prime-coat

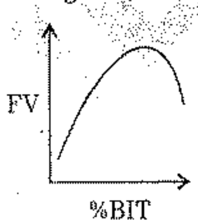
Codes:

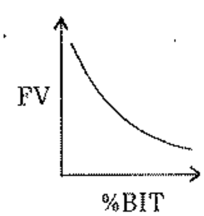
	A	**B**	**C**	**D**
(a)	3	4	1	2
(b)	2	1	4	3
(c)	3	1	4	2
(d)	2	4	1	3

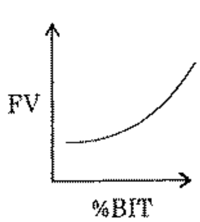
21. What are the standards for testing of road macadam in Aggregate Impact Test?

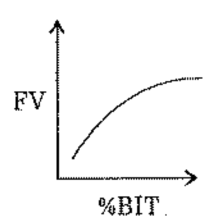
(a) 14 kg wt, 38 cm drop, 15 blows
(b) 14 kg wt, 35 cm drop, 20 blows
(c) 18 kg wt, 35 cm drop, 15 blows
(d) 18 kg wt, 30 cm drop, 20 blows

22. Which one of the following diagrams illustrates the relationship between flow value (FV) and percentage bitumen (%BIT)?

(a) 

(b) 

(c) 

(d) 

23. Consider the following statements related to Los Angeles Abrasion test on aggregates:
    1. It evaluates hardness of source-rock.
    2. It has a coefficient of variation of about 30 per cent.

    Which of these statements is/are correct?

    (a) 1 only (b) 2 only
    (c) Both 1 and 2 (d) neither 1 nor 2

24. Which one of the following criteria is used for obtaining the value of modulus of subgrade reaction from the plate bearing test data?
    (a) Slope of pressure settlement graph.
    (b) Pressure corresponding to the settlement of 1.25 mm.
    (c) Pressure corresponding to a pressure of 1.25 kg/cm^2.
    (d) Pressure corresponding to the settlement of 1.50 mm.

25. Emulsion is used as a binder in which of the following stages of construction?
    1. Surface dressing work
    2. Sealing open textured surfacing
    3. Filling cracks in pavement
    4. Prime-coat
    5. Pre-coating of aggregates

    Select the correct answer the using the codes given below:

    (a) 1, 2, 3 and 5 (b) 2, 3, 4 and 5
    (c) 1, 2 and 4 (d) 1 and 3 only

26. **Assertion (A):** California Bearing Ratio Test is carried out to evaluate the stability of soil sub grade and other flexible pavement materials.
    Reason (R): It is essential that at no time are the soil sub grade as well as other flexible pavement materials over stressed.

27. Which one of the following tests is performed in the laboratory to determine the extent of weathering of aggregates for roadworks?
    (a) Soundness test (b) Crushing test
    (c) Impact test (d) Abrasion test

28. In a bituminous concrete mix, bitumen and aggregates by weight are 5% and 95% respectively. Specific gravity of bitumen and aggregates are 1 and 2.5 respectively. Theoretical unit weight of the mix will be
    (a) 2857 kg/m^3 (b) 2630 kg/m^3
    (c) 2325 kg/m^3 (d) 2208 kg/m^3

29. Match **List-I** (Property of aggregate) with **List-II** (Test associated with the property) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Strength	**1.** Impact
B. Hardness	**2.** Soundness
C. Toughness	**3.** Stripping
D. Durability	**4.** Crushing
	5. Abrasion

Codes:

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

30. Bituminous materials are used in highway construction primarily because of their
- (a) cementing and water-proofing properties
- (b) load bearing capacity
- (c) high specific gravity
- (d) black colour which facilitates road marking

31. While using the data from a plate bearing test for determining the modulus of sub-graded reaction, the value of settlement to be used is
- (a) 12.5 mm
- (b) 6.5 mm
- (c) 4.0 mm
- (d) 1.25 mm

32. Match **List-I** (Tests) with **List-II** (Physical properties) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Abrasion test	**1.** Durability
B. Crushing strength test	**2.** Toughness
C. Impact test	**3.** Hardness
D. Soundness test	**4.** Compressive strength

Codes:

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

33. If the load carried by a CBR specimen at 2.5 mm penetration is 35 kg/cm^2, the CBR of the soil is
- (a) 10%
- (b) 35%
- (c) 50%
- (d) 70%

34. What is the recommended grade of tar for grouting purpose?
- (a) RT-I
- (b) RT-2
- (c) RT-3
- (d) RT-5

35. Match **List-I** (Material) with **List-II** (Property) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Asphalt	**1.** Coal distilled in the absence of air
B. Cutback bitumen	**2.** Bitumen dissolved in aqueous medium
C. Bitumen emulsion	**3.** Bitumen with volatile diluent
D. Tar	**4.** Bitumen along with some proportion of minerals

Codes:

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

36. Penetration to know bitumen grade is measured in
(a) one-hundredth of mm (b) one-tenth of mm
(c) one-tenth of an inch (d) one micron

37. What does penetration grade of bitumen 80/100 indicate?
(a) Dynamic viscosity (b) Kinematic viscosity
(c) Ductility (d) Hardness

38. Match **List-I** (Test) with **List-II** (Purpose) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Spot test	**1.** To assess the resistance of aggregates to weathering action
B. Impact test	**2.** To test the hardness property of stones
C. Soundness test	**3.** For detecting overheated or cracked bitumen
D. Abrasion test	**4.** To test the resistance of the aggregates to fracture

Codes:

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

39. What should be the relative magnitude of free carbon in bitumen over that in tar for road construction?
(a) More (b) Less
(c) Equal (d) Unrelative

40. The modulus of subgrade reaction is evaluated from which one of the following?
(a) Plate-bearing test (b) CBR test
(c) Direct shear test (d) Triaxial test

41. What total weight of specimen is used for a penetration test on bitumen when the Penetrometer consists of a needle assembly?
(a) 1 gm (b) 10 gm
(c) 100 gm (d) 200 gm

42. In the Marshall method of mix design, the coarse aggregates, fine aggregates, filler and bitumen, having respective specific gravities of 2.62, 2.72, 2.70 and 1.02 are mixed in the ratio of 55, 34.6, 4.8 and 5.6 per cent, respectively. The theoretical specific gravity of the mix would be
(a) 2.36 (b) 2.40
(c) 2.44 (d) 2.50

43. The plate load test conducted with a 75 cm diameter plate on soil subgrade yielded a deflection of 2.5 mm under a stress of 800 N/cm^2. The modulus of elasticity of the subgrade soil, in kN/cm^2 is

(a) 141.6 (b) 154.6
(c) 160.0 (d) 185.4

44. **List-I** below gives a list of physical properties of aggregates which should be determined to judge their suitability in road construction. **List-II** gives a list of laboratory tests which are conducted to determine these properties. Match **List-I** with **List-II** and select the correct answer from the codes given below the lists:

List-I	List-II
A. Hardness	**1.** Water adsorption
B. Porosity	**2.** Impact test
C. Toughness	**3.** Soundness test
D. Durability	**4.** Abrasion test

Codes:

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

45. A Marshall specimen is prepared for bituminous concrete with a bitumen content of 5 per cent by weight of total mix. The theoretical and the measured unit weights of the mix are 2.442 g/cm^3 and 2.345 g/cm^3, respectively. The bitumen has a specific gravity of 1.02. The per cent voids in mineral aggregate filled with bitumen (VFB) are

(a) 34.55 (b) 35.9
(c) 73.55 (d) 74.3

46. **List-I** contains some properties of bitumen. **List-II** gives a list of Laboratory Tests conducted on bitumen to determine the properties. Match the property with the corresponding test and select the correct answer using the codes given below the lists:

List-I	List-II
A. Resistance to flow	**1.** Ductility test
B. Ability of deform under load	**2.** Penetration test
C. Safety	**3.** Flash and fire point test

Codes:

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

47. If aggregate size of 50–40 mm is to be tested for finding out the portion of elongated aggregates using length gauge, the slot length of the gauge should be

(a) 81 mm (b) 45 mm
(c) 53 mm (d) 90 mm

48. A subgrade soil sample was tested using standard CBR apparatus and the observations are given below.

Load, kg	Penetration, mm
60.5	2.5
80.5	5.0

Assuming that the load-penetration curve is convex throughout, the CBR value 9%) of the sample is

(a) 6.5
(b) 5.5
(c) 4.4
(d) 3.9

49. The consistency and flow resistance of bitumen can be determined from which of the following?

(a) Ductility test
(b) Penetration test
(c) Softening point test
(d) Viscosity test

50. The specific gravity of paving bitumen as per **IS:73-1992** lies between

(a) 1.10 and 1.06
(b) 1.06 and 1.02
(c) 1.02 and 0.97
(d) 0.97 and 0.92

51. A combined value of flakiness and elongation index is to be determined for a sample of aggregates. The sequence in which the two tests are conducted is

(a) elongation index test followed by flakiness index test on the whole sample
(b) flakiness index test followed by elongation index test on the whole sample
(c) flakiness index test followed by elongation index test on non-flaky aggregates
(d) elongation index test followed by flakiness index test on non-elongated aggregates

52. During a CBR test, the load sustained by a remoulded soil specimen at 5.0 mm penetration is 50 kg. The CBR value of the soil will be

(a) 10.0%
(b) 5.0%
(c) 3.6%
(d) 2.4%

ANSWERS

1.	(a)	2.	(b)	3.	(a)	4.	(c)	5.	(b)	6.	(d)	7.	(c)
8.	(d)	9.	(c)	10.	(d)	11.	(a)	12.	(a)	13.	(b)	14.	(c)
15.	(d)	16.	(d)	17.	(d)	18.	(b)	19.	(c)	20.	(b)	21.	(a)
22.	(c)	23.	(a)	24.	(b)	25.	(a)	26.	(c)	27.	(a)	28.	(c)
29.	(a)	30.	(a)	31.	(d)	32.	(a)	33.	(c)	34.	(d)	35.	(d)
36.	(b)	37.	(d)	38.	(a)	39.	(b)	40.	(a)	41.	(c)	42.	(c)
43.	(a)	44.	(b)	45.	(c)	46.	(a)	47.	(a)	48.	(c)	49.	(d)
50.	(c)	51.	(b)	52.	(d)								

HINTS AND SOLUTIONS

12. (a)

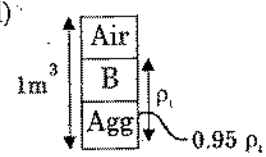
Sol.
$$k_1a_1 = k_2a_2$$
$$\Rightarrow 10\times 30 = k_2 \times 75$$
$$k_2 = \frac{300}{75} = 4$$

15. (d)

Sol.
$$k_1a_1 = k_2a_2$$
$$\Rightarrow k_2 = \frac{k_1a_1}{a_2} = \frac{200\times 30}{75} \quad [k_2 = 80]$$

33. (c)

Sol. (Assuming No Air void)



$$\Rightarrow 1 = \frac{0.05\rho_t}{1\times\rho_\omega}$$

$$\therefore \rho_t = \frac{100}{\left(0.05+\frac{0.95}{2.5\rho_\omega}\right)} = \frac{100}{\left(0.05+\frac{0.95}{2.5}\right)}$$

$$= 2325.58 \text{ kg/m}^3$$

37. (d)

Sol.
$$k = \frac{P}{\Delta}$$

where $\Delta = 0.125$ cm $= 1.25$ mm

39. (c)

Sol.
$$\text{CBR} = \frac{\frac{35}{1370}}{\frac{\pi}{4}(5)^2}\times 100 = 50\%$$

40. (d)

Sol. For grouting purpose RT-5 is used.

42. (b)

Sol. Penetration to know bitumen grade is measured in $\frac{1}{10}$ of mm

52. (c)

Sol.
$$G_t = \frac{(W_1+W_2+W_3+W_4)}{\left(\frac{W_1}{G_1}+\frac{W_2}{G_2}+\frac{W_3}{G_3}+\frac{W_4}{G_4}\right)}$$

$$= \frac{(55+34.6+4.8+5.6)}{\left(\frac{55}{2.62}+\frac{34.6}{2.72}+\frac{4.8}{2.70}+\frac{5.6}{1.02}\right)} = 2.44$$

53. (a)

Sol. We know that, for rigid pavement

$$\Delta = \frac{1.18pa}{E_s}$$

$$\therefore E_s = \left(\frac{1.18\times 0.8\times 37.5}{0.25}\right) = 141.6 \text{ kN/cm}^2$$

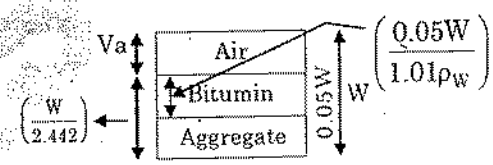
55. (c)

Sol. We know that

$$\rho_w G_t = \left(\frac{W}{V_s}\right)$$

$$\rho_w G_m = \left(\frac{W}{V_s+V_a}\right) = 2.345$$

$$V_s + V_a = \frac{W}{2.345\rho_w}$$



$$\Rightarrow V_a = \left(\frac{W}{2.345\rho_w} - \frac{W}{2.442\rho_w}\right)$$

$$\text{V.F.B.} = \frac{V_{bit}\times 100}{(V_{air}+V_{bit})}$$

$$= \frac{\left(\frac{0.05W}{1.05\rho_w}\right)\times 100}{\frac{W}{2.345\rho_w} - \frac{W}{2.442\rho_w} + \frac{0.05W}{1.01\rho_w}}$$

$$= \frac{\frac{0.05}{1.02}}{0.0664} = 73.78\%$$

58. (c)

Sol. Generally C.B.R. value at 2.5 mm penetration is higher than 5.0 mm penetration

$$\therefore \text{C.B.R.} = \left(\frac{60.5}{1370}\times 100\right) = 4.4\ \%$$

63. (d)

Sol. Standard load for 5.00 mm penetration = 2055kg.

$$\therefore \text{CBR} = \left(\frac{50}{2055}\times 100\right) = 2.4\%$$

CHAPTER

5

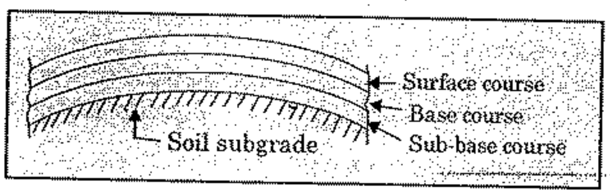
Pavement Design

TYPES OF PAVEMENT

A pavement is the load bearing and load-distributing component of a road. Pavements can be classified as:

(a) Flexible (b) Rigid (c) Semi-rigid (d) Composite

(a) Flexible Pavement



Flexible Pavement

- A flexible pavement is one that is made up of one or more layers of materials, the highest quality material forming the top layer.
- Loads are transmitted through the layers, care being taken to ensure that the stresses in each layer are within the permissible values and the stress on the sub-grade is within its bearing power.
- The load carrying capacity of the flexible pavement is derived from the load-distribution property and not from its flexural or bending strength.
- The flexible pavement layers reflect the deformation of the lower layer, Thus if the lower layer of the pavement of soil subgrade is undulated, the flexible pavement surface also gets undulated.
- A typical flexible pavement consists of four components

  (i) Soil subgrade (ii) Sub base course (iii) Base course (iv) surface course

  Soil subgrade : It is a layer of natural soil prepared to receive stress from layers above. It is normally the top natural soil.

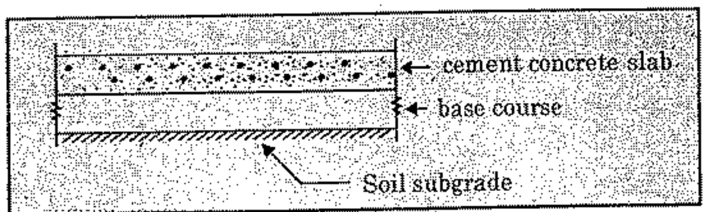
  Sub base course : It is provided beneath base course. Primary function is to provide structural support. improve drainage and reduce intrusion of fines from subgrade in the pavement structure. A pavement constructed over a high quality stiff subgrade does not require subgrade.

  Base course : It is provided immediately below the surface course, provides load distribution & contributes to sub-surface drainage. It is composed of crushed stones.

  Surface course : It is directly in contact with traffic load. It is superior in quality.Its function is to provide friction, smoothness and drainage. It is made up of dense graded asphaltic concrete.
- The flexible pavement layer transmit the vertical or compressive stresses to the lower layer by grain to grain transfers through the points of contact in the granular structure

- Bituminous concrete is one of the best flexible pavement layer materials, other material which fall under this group are, all granular materials with or without binders, granular base and sub base course materials like the water bound macadam, crushed aggregate, gravel, soil aggregate mixes etc.
- Major pavement failure are fatigue cracking, rutting and thermal cracking. Rutting is the depression in localised area. IRC considers fatigue cracking and rutting for flexible pavement design.

(b) Rigid Pavement



Rigid Pavement

- A rigid pavement (constructed with cement concrete slabs) depends upon the flexural strength or beam action of the slab for withstanding the wheel load.
- Thus, a major contributor to the load-bearing capacity is the slab itself.
- The stresses are not transferred from grain to grain to the lower layer as in case of flexible pavement layers.
- The rigid pavement made of portland cement concrete either plain, reinforced, prestressed concrete. The plain cement concrete slabs are expected to take up about 40 kg/cm^2 flexural stress.
- The main point of difference in the structural behaviour of rigid pavement an compared to the flexible pavement is that, the critical condition of stress in the rigid pavement is the maximum flexural stress occurring in the slab due to wheel load and the temperature changes. Whereas in the flexible pavement it is the distribution of compressive stress.
- The cement concrete pavement slab can very well serve as a wearing surface as well an effective base course.
- The rigid pavement are usually designed and the stress are analyzed using the elastic theory, assuming the pavement as an elastic plate resting over an elastic or viscous foundation.
- Major pavement failure are fatigue cracking and pumping. Normally in India Fatigue cracking is the only criteria adopted for rigid pavement design. Allowable no. of road repetition to cause fatigue cracking depends on stress ratio. Where stress ratio is the ratio of flexural tensile stress & concrete modulus of rupture.
- Pumping is the ejection of soil slurry through joints and cracks of cement concrete pavement causing downward movement of slab under heavy load.

(c) Semi-rigid

- A Semi-rigid pavement represents the intermediate stage between the flexible and the rigid types.
- It derives strength both by load-spreading and flexural action.
- When bonded materials like the pozzolanic concrete (lime-flyash-aggregate mix), lean cement concrete or soil cement are used in the base course or sub base course layer the pavement layer has considerably higher flexural strength than the common flexible pavement layers.

(d) Composite

- A composite pavements has a mixture of the above types in its layers. One example is a pavement consisting of lean concrete base, a roller compacted concrete slab over it and a surfacing of bituminous concrete.

FUNCTIONS OF PAVEMENT COMPONENTS

1. Soil subgrade

- The pavement load is ultimately taken by soil subgrade. Hence, in no case it should be over stressed and 50 cm layer of soil subgrade should be well compacted at O.M.C.
- Common strength tests used for evaluation of soil subgrade are:
  (i) CBR test
  (ii) California resistance value test
  (iii) Triaxial compression test
  (iv) Plate bearing test.

2. Sub base and base course

- These are composed of broken stone aggregates. It is desirable to use smaller size graded aggregates at sub base course instead of boulder stones.
- Base and sub base courses are used under flexible pavements primarily to improve load supporting capacity by distribution of the load through a finite thickness.
- Base courses are used under rigid pavements for
  (i) preventing pumping (ii) protecting the subgrade against frost action.

3. Wearing Course

- Purpose of this course is to give smooth riding surface. It resists pressure exerted by tyres and take up wear and tear due to traffic. It also offers water tightness.
- The stability of wearing course is estimated by Marshall stability test where in optimum % of bituminous material is worked out based on stability density, VMA & VFB. Plate bearing test and Banklemann beam test are also some times made use of for evaluating the wearing course and the pavement as a whole.

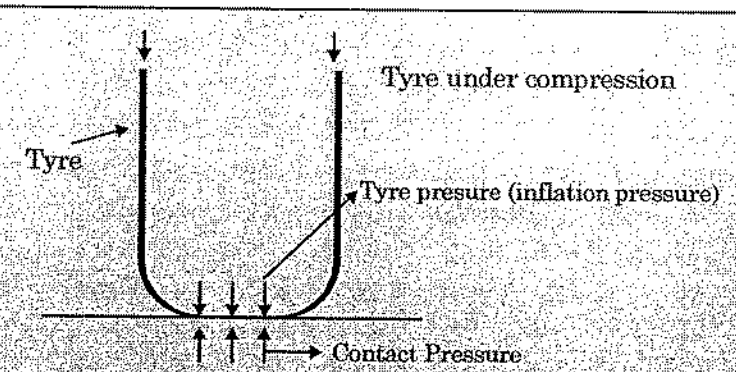
VARIOUS TERMS & DESIGN CONCEPT

- For geometrical design we need to consider all types of vehicles but for pavement design, only vehicles having significantly heavy loads are important. These vehicles are generally commercial vehicles. For pavement design we need to count only commercial vehicles.
- As per IRC, vehicles having gross load greater than **3 ton** are called commercial vehicles.

Load Transferred through wheels

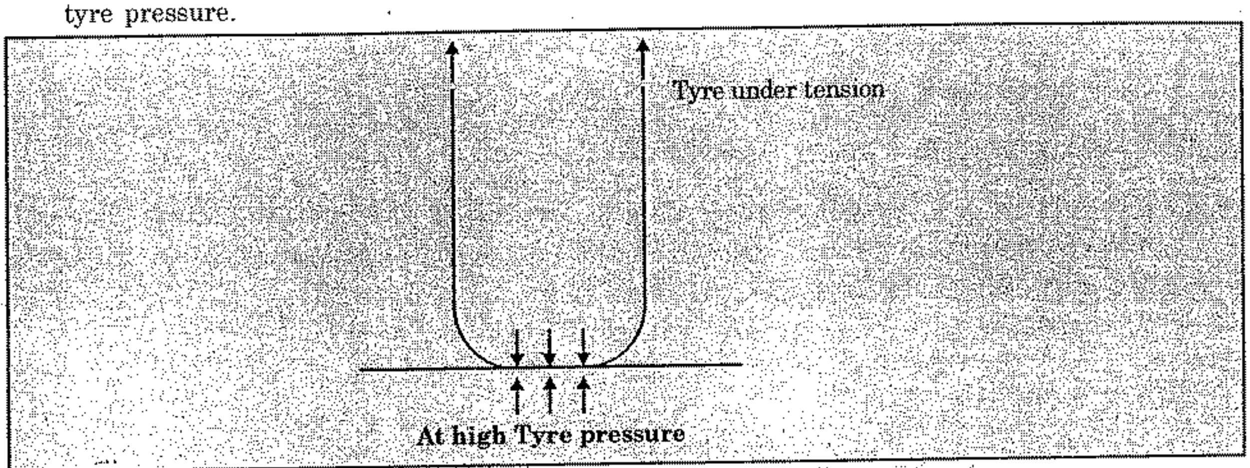
- While analysing load transfer through wheels we need to consider total wheel load, contact area and distribution of pressure over contact area.

Contact Pressure and Tyre Pressure.



At low tyre pressure

- At low tyre pressure the tyre comes under compression. Hence contact presure is greater than tyre pressure.



- At high tyre pressure, tyre come under tension. Hence contact pressure is less than tyre pressure.

$$\text{Rigidity factor} = \frac{\text{Contact pressure}}{\text{Tyre pressure}}$$

- Generally for design purpose.

$$\text{Tyre Pressure} = 0.7 \text{ Mpa, RF} = 1$$
$$\text{Tyre Pressure} > 0.7 \text{ MPa, R.F.} < 1$$
$$\text{Tyre Pressure} < 0.7 \text{ MPa, R.F.} > 1$$

$$\text{Contact Pressure} = \frac{\text{Wheel load}}{\text{Area of imprint}}$$

- The imprint area is generally taken as circular area for design purpose.

Design Load Consideration

Following effects are considered while computing the design load.

- The traffic volume in each year will increase on the road
- different vehicles will have different weight,
- wheel loads are applied over different portion of pavement not at a single location.

Traffic Forecast

- In this case we estimate number of commercial vehicles that that is going to utilize the road over the design life.

  N = Commulative no. of commercial vehicle during design life = $\frac{365A\left[(1+r)^n - 1\right]}{r}$

  A = Initial design traffic in vehicle per day in the year of completion of construction.

  $A = P(1+r)^x$

  r = Rate of increase expressed in fraction normally taken as 7.5% i.e. 0.075 per year.
  P = No. of commercial vehicle per day at last count.
  x = Construction period in year
  n = Design period [Design life].

- The commulative no. of vehicles so obtained during design life above needs to be modified for
  (a) Directional distribution of traffic.
  (b) Lateral placement characteristic of wheel on pavement.
  (c) Load spectrum (empty, heavy etc.)

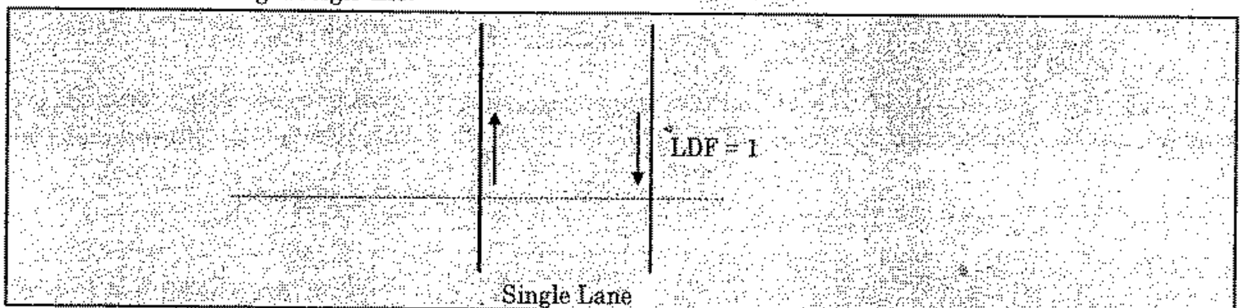
Directional distribution of wbeel Loads

- All vehicles do not move on same path i.e. do not take same lateral distribution. Thus, all vehicles do not load the road at same point. Thus we take only certain percentage of cumulative traffic as design traffic.

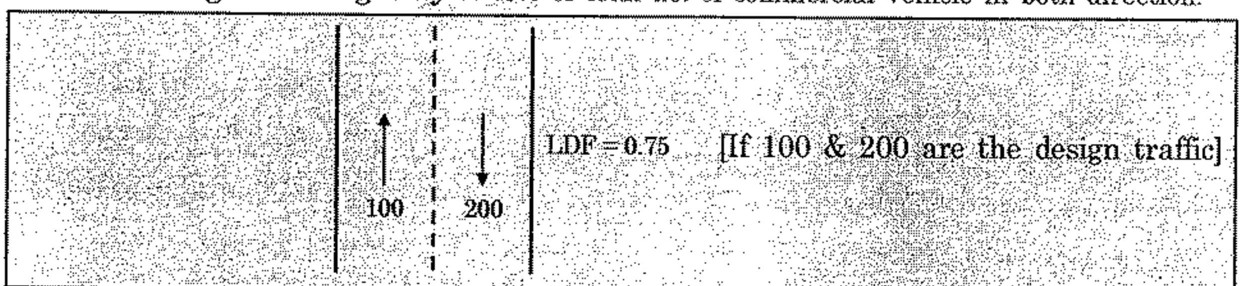
IRC recommendation for lateral distribution factor

Single lane road

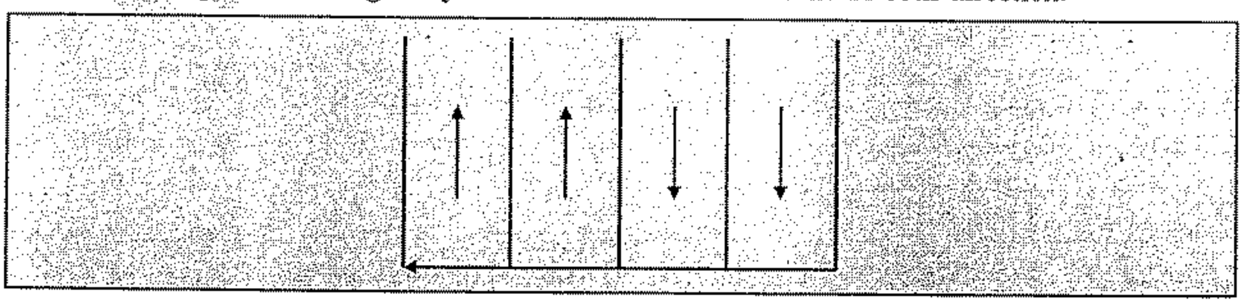
- In this case, design is based on total no. of vehicles in both direction i.e. lateral distribution factor = 1.
- Hence, Design traffic = N × lateral distribution factor, where N = Total no. of vehicles in both direction during design life.



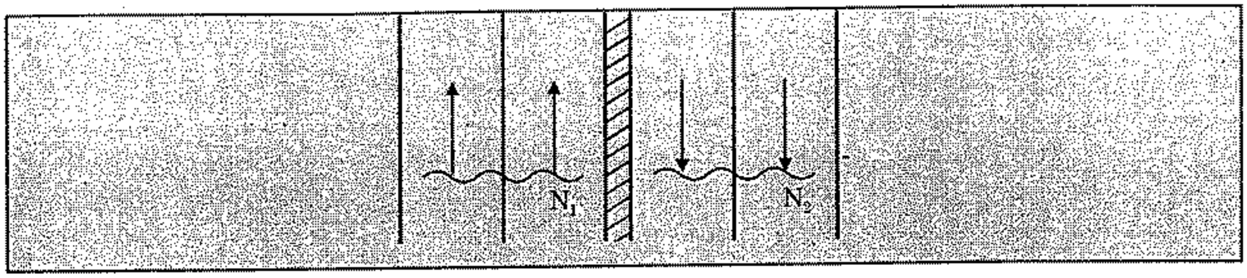
Two lane single carriageway : 75% of total no. of commercial vehicle in both direction.



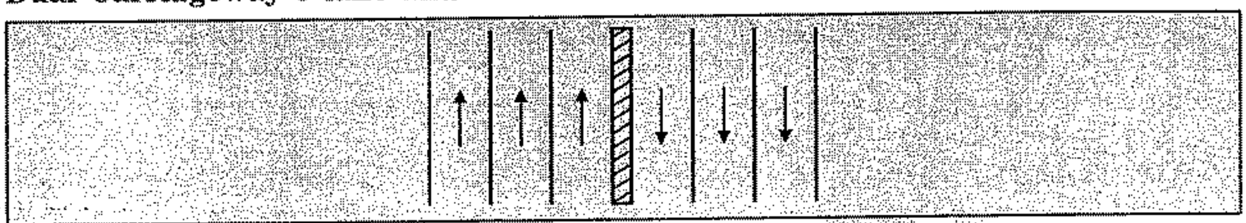
Four lane signal carriageway : 40% of total number traffic in both direction.



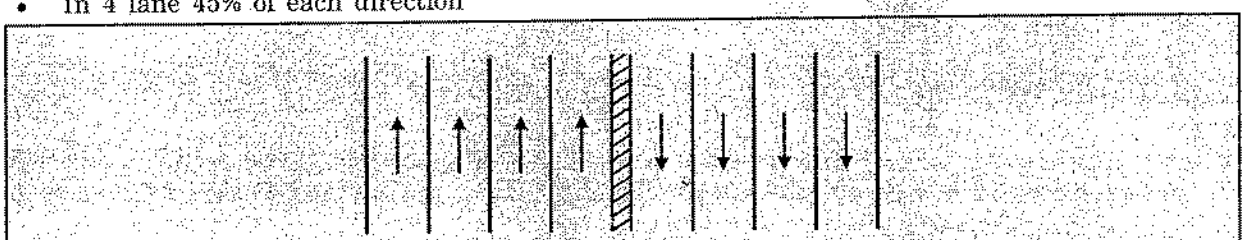
Dual Carriageway two lane [Two lane] : 75% of no. of vehicle in each direction is taken as design traffic. Design traffic = 0.7 $\{\max(N_1, N_2)\}$



Dual Carriageway 3 lane and 4 lane : 60% of each direction in 3 lane.



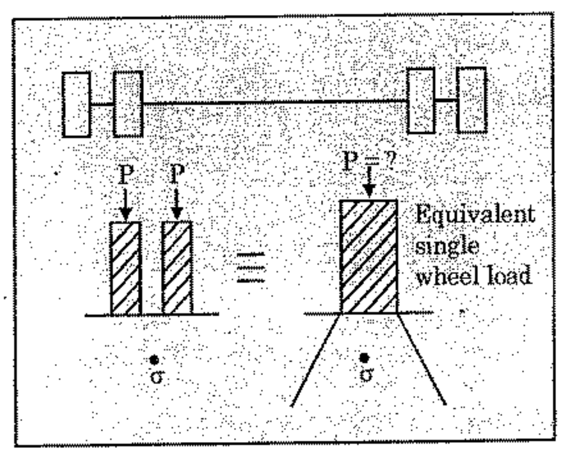
- In 4 lane 45% of each direction



Traffic LOAD Consideration in Design

- We generally use three approaches for the design of the pavement.
  (a) Fixed traffic approach
  (b) Fixed vehicle approach
  (c) Variable traffic and variable vehicle approved.

(a) Fixed Traffic Approach

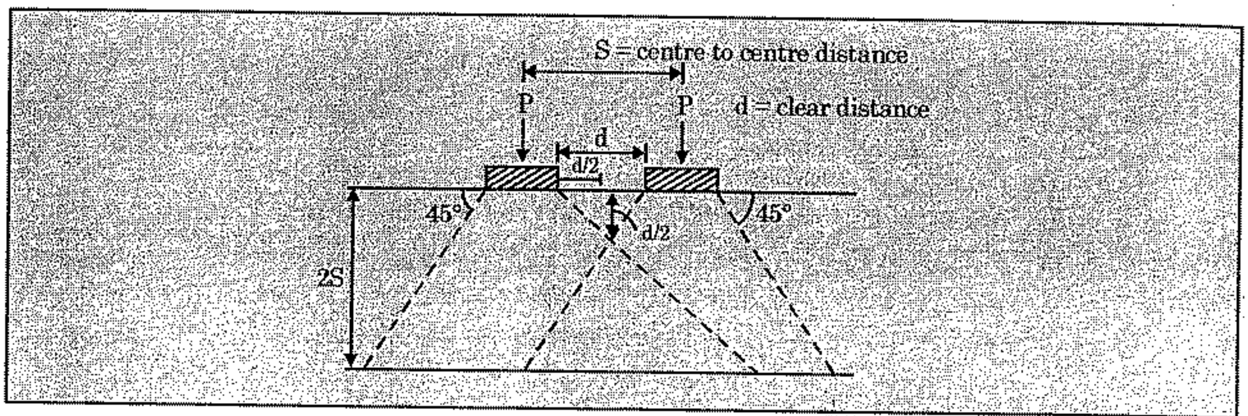


- Heaviest vehicle is considered in this.
- Number of repetitions not considered.
- Pavement is designed for single wheel-load
- Multiple wheel loads are converted to equivalent single wheel load [ESWL].
- In the case of Airport pavement design, we can consider this approach because one single load movement is sufficient to cause damage to pavements. However, for highway pavements, we will not use this approach because in that case, it is the no. of repetition of smaller loads which will cause the damage to pavement.

Equivalent Single Wheel Load

- It is defined as load on the single tyre which will cause an equivalent magnitude of pre-selected parameters (stress, strain, deflection, distress etc.) at a given location to that resulting from the multiple wheel load at the same location.

- In our course we will consider equivalency in terms of stress.



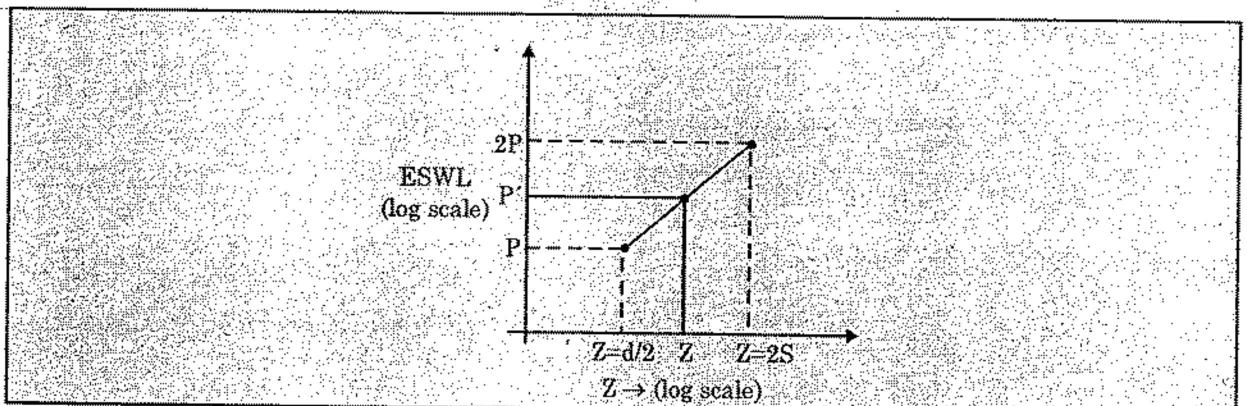
- Upto a depth of d/2 no stress overlap occurs. Hence equivalent single load.

$$ESQL = P$$

- At depth greater than equal to 2S, the effect of overlap is such that we can consider

$$ESWL = 2P$$

- For intermediate location, it is assumed that linear relationship exist between the equivalent load and depth plotted on log-log paper.

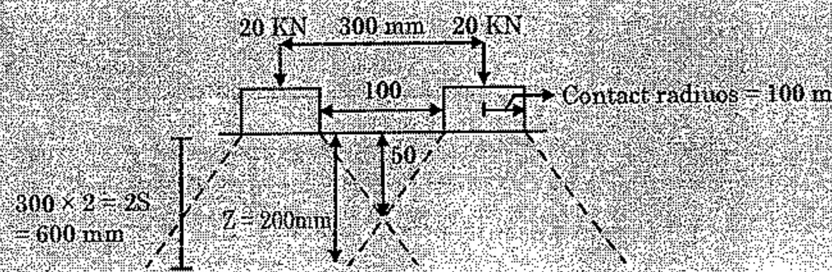


$$\boxed{\log P' = \log P + \frac{\log 2P - \log P}{\log 2S - \log \frac{d}{2}} \times \left(\log Z - \log \frac{d}{2}\right)}$$

$$\boxed{\log P' = \log P + \frac{\log Z}{\log \frac{4S}{d}} \times \log\left(\frac{2Z}{d}\right)}$$

Example 1

For the multiwheel condition shown below find ESWL at Z = 200 mm.



Sol.

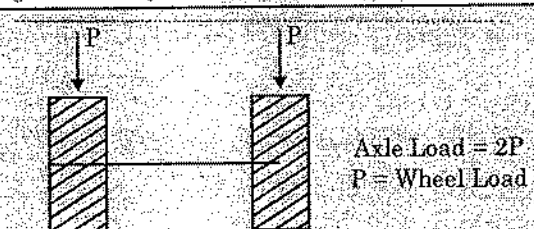
$$\log(\text{ESQL}) = \log 20 + \frac{\log 40}{\log 600 - \log 50}\left[\log(200) - \log\frac{50}{2}\right]$$

$$\text{logg (ESWL)} = 1.46734$$

$$\Rightarrow \quad \text{ESWL} = 10^{1.46734} = 29.444 \text{ kN.}$$

(b) Fixed Vehicle Approacb

- In this case design is governed by number of repetations of standard axle.



- 80 kN single axle is considered to be standards axle load.
- Axle's that are not either signal or not equal to 80 KN are converted to equivalent number of standards axle load using equivalent axle load factor [EALF].
- Equivalent axle load factor defines the damage caused to the pavement by one application of the axle load under consideration relative to the damage caused by single application of a standard axle load (80 kN).
- We multiply the repetation of a given axle load by EALF to obtain equivalent number of 80 KN axle load repetation
- Sum of the equivalent repetations obtained from all the axle load anicipitated (during design life) is used as a design parameter.
- This approach is most commonly used approach for highway.

Total no. of repetition of equivalent standard axle load = $\sum_{i=1}^{m} F_i n_i$

m = number of axle load groups

F_i = EALF for i^{th} group.

n_i = number of application of i^{th} group during design period.

For example, in the given data, m = 3

	axle load range (KN)	Mean of axle load (KN)	No. of vehicles (n_i)
1.	50 – 80	65	5
2.	80 – 100	90	6
3.	100 – 120	110	7

- Equivalent Axle Load Factor (EALF) is generally taken as

$$\boxed{\text{EALF of a (axle load)} = \left(\frac{\text{axle load}}{\text{standard axle load}}\right)^4}$$ [Fourth Power Law]

For example: 160 KN axle load has EALF= $\left(\frac{160}{80}\right)^4 = 16$

Example 2

The result of 1day axle load survey of trucks on a road is tabulated as below : Find the no. of repetitions of a standard 80 KN axle in a year.

Wt in KN	Frequency (n)	Mid Point (x)	EALF = $\left(\frac{x}{80}\right)^4$	n_i EALF
0 - 40	50	20	$\frac{1}{256}$	$\frac{50}{256}$
40 - 80	250	60	$\frac{81}{256}$	$\frac{(250\times81)}{256}$
80 - 120	400	100	$\frac{625}{256}$	$\frac{(400\times625)}{256}$
120 - 160	25	140	$\frac{2401}{256}$	$\frac{(250\times2401)}{256}$
				$\Sigma = 3400$

No. of repetition of standard axles in one year will be 3400 × 365 = 1.241 MSA

Example 3

If in the previous example, 950 no. of axles were surveyed and the no. of vehicles were 400, find the vehicle damage factor.

Sol. 400 vehicle is equivalent to 3400 nos. standard axle,

1 vehicle is equivalent to $\frac{3400}{400} = 8.5$ standard axle. Thus vehicle damage factor = 8.5

$\Rightarrow$ 1 commercial vehicle = 8.5 no.of standards axles.

Thus vehicle damage factor = 8.5

Note : VDF for a road traffic is calculated using axle load survey.

Example 4

The no. of commercial vehicle per day at present count is 6000. Design life is 15 yrs. Traffic growth rate is 8%. VDF is 4.5. lateral distribution factor for 6 lane divided highway = 0.6. Calculate the no. of standards axles in the design life if the construction period is 2 yrs.

Sol. P = 6000 vehcle/day

Traffic after two year = $P(1+r)^x$

$$(A) = 6000(1 + 0.08)^2$$
$$= 6000(1.08)^2$$
$$= 6998 \text{ veh./day}$$

$\Rightarrow$ No. of commercial vehicles after the end of construction perios is 6998 veh./day

No. of commercial vehicle in design life

$$= N = \frac{365A\left[(1+r)^n - 1\right]}{r}$$

$$= \frac{365 \times 6998\left[(1.08)^{15} - 1\right]}{0.08}$$

$$N = 69.35 \times 10^6 \text{ veh.}$$

If lateral destribution of traffic is accounted for then

No. of traffic in design life × lateral distribution factor = $69.35 \times 10^6 \times 0.6 = 41.61 \times 10^6$

If damaging potential of vehicle is accounted for then,

1 Veh. = 4.5 no. of standard axle.

No. of standards axle in design life = $41.61 \times 10^6 \times 4.5$

$$= 187.245 \times 10^6 \text{ axle}$$
$$= 187.245 \text{ MSA}$$

Note : No. of standard axle in design life to be adopted for design is taken as

$$N = \frac{365ADF\left((1+r)^n - 1\right)}{r}$$

When A = no. of commercial vehicle/day after the end of construction.

D = Vehicle damage factor

F= Lateral distribution factor

r = growth rate

n = design life in years.

Example 5

Calculated equivalent single wheel load of a dual wheel assembly carrying 2050 kg, each for pavement thickness of 15, 20, and 25 cm. Centre to centre spacing between tyre is 30 cm. distance between adjacent wall of tyre in 12 cm.

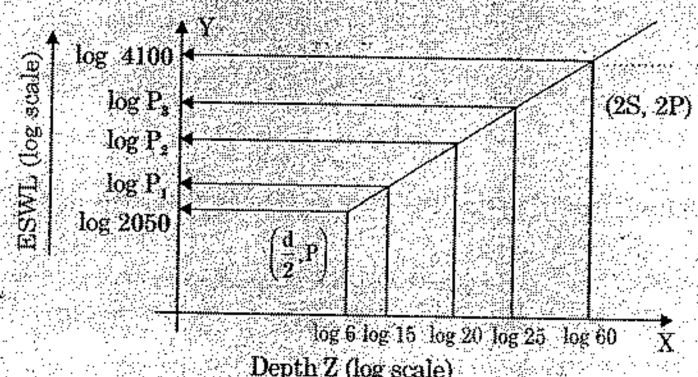
Sol. For dual wheel assembly

$$P = 2050 \text{ kg} ; \; S = 30 \text{ cm} ; \; d = 12 \text{ cm}$$

We have to find ESWL @ Z = 15 cm, 20 cm, 25 cm

$$\therefore \quad \text{ESWL} @ \frac{d}{2} = \frac{12}{2} = 6 \text{ cm} ; \; P = 2050 \text{ kg}$$

$$\therefore \quad @2S = 2 \times 30 = 60 \text{ cm}, \; 2P = 4100 \text{ kg}$$



Corresponding to depth, Z = 15 cm

$$\Rightarrow \quad \log P_1 - \log 205 = \frac{\log 4100 - \log 2050}{(\log 60 - \log 6)}(\log 15 - \log 6)$$

$$\therefore \quad \log P_1 = 3.431$$

$$\therefore \quad P_1 = 2701.13 \text{ kg}$$

Corresponding to depth Z = 20 cm

$$\log P_2 - \log 2050 = \frac{(\log 4100 - \log 2050)}{(\log 60 - \log 6)}(\log 20 - \log 6)$$

$$\Rightarrow \quad \log P_2 = 3.4691$$

$$\Rightarrow \quad P_2 = 2945.48 \text{ kg}$$

corresponding to depth z = 25 cm

$$\log P_3 - \log 2050 = \frac{\log 4100 - \log 2050}{(\log 60 - \log 6)}(\log 25 - \log 6)$$

$$\log P_3 = 3.4983$$

$$P_3 = 3150 \text{ kg}$$

Pavement Thickness cm	ESWL (kg)
15	2701.13
20	2945.48
25	3150.13

Example 6

The results of an one-day axle load survey of trucks on a road are tabulated in table. Determine the number of repetitions of a standard 80 kN axle in a year.

Weight in tonnnes	*Mid-point in tonnes*	*Mid-point in kN (L)*	*Number of axles (N)*	$\left(\frac{L}{80}\right)^4$	$\left(\frac{L}{80}\right)^4 \times N$
1—2	1.5	15	14	0.0012	0.02
2—3	2.5	25	76	0.0095	0.72
3—4	3.5	35	77	0.0366	2.82
4—5	4.5	45	70	0.1001	7.01
5—6	5.5	55	28	0.2234	6.26
6—7	6.5	65	18	0.4358	7.84
7—8	7.5	75	10	0.7725	7.72
8—9	8.5	85	11	1.2744	14.02
9—10	9.5	95	11	1.9885	21.87
10—11	10.5	105	11	2.9675	32.64
11—12	11.5	115	12	4.2700	51.24
12—13	12.5	125	15	5.9605	89.41
13—14	13.5	135	7	8.1021	56.76
14—15	14.5	145	3	10.7922	32.38
15—16	15.5	155	1	14.0918	14.09
Total			364		344.8

Number of 80 kN axles per year = 344.8 × 365 = 128852

METHOD OF PAVEMENT DESIGN

Various approach for flexible pavement design may be classified in a three broad groups.

(a) Empirical Methods

- These are based on physical properties and strength parameter of soil subgrade.
- The group index method, CBR method Stabilometer method and Mc-lead method etc. are empirical methods.

(b) Semi Empirical or semitheoretical method: These methods are based on stress Strain function and experience. *e.g.*, Triaxial test method.

(c) Theoretical Methods :These are based on mathematical computation for *e.g.*, Burmister method is based on elastic two layer theory.

GROUP INDEX METHOD

- Group index value is an arbitrary index assigned to the soil types in numerical equations based on the percent fines, liquid limit and plasticity index.
- The G.I value of soils vary in the range of 0 to 20.
- The higher the G.I value weaker the soil subgrade hence greater thickness of Pavement required.

$$\text{G.I} = (0.2\,a + 0.005\,ac + 0.01\,bd) \quad \text{... (i)}$$

where,

$a = p - 35 \not> 40$ (expressed as whole no between 0 - 40)

$b = p - 15 \not> 40$ (expressed as whole no between 0 - 40)

$c = w_L - 40 \not> 20$ (expressed as whole no between 0 - 20)

$d = I_P - 10 \not> 20$ (expressed as whole no between 0 - 20)

p = percentage finer (percentage of soils passing from 0.075 mm seive)

w_L = Liquid limit

I_P = plasticity index = $(w_L - w_P)$

- To design the pavement thickness by this method, first the G.I value of the soil is found. The anticipated traffic is estimated and is designated as light, medium or heavy.

Traffic volume (commercial vehicle)	No. of vehicle per day (Aticipated Traffic)
Light	Less than 50
Medium	50 to 300
Heavy	over 300

- Based on the anticipated traffic and group index value, thicknes of pavement layer is calculated. The thickness of sub-base depends only on gorup index value. However the thickness of surfae and base course combined depends on both the traffic as well as GI value.

$$\left.\begin{array}{c}\text{Surface}\\ \text{Base}\end{array}\right\} f(\text{Traffic, GI})$$

$$\left.\text{Sub base}\right\} f(\text{GI})$$

Subgrade

- In the exam a table would be given. From the table total thickness corresponding to GI value can be calculated.

Note : In actual practices we have curves available for total combined thickness and thickness of surface and base course only corresponding to traffic and GI value. Hence from total thicknes, thickness of sub-base is obtained by deducting, the thickness of (surface + Base course) from Total thickness.

Limitation

(1) G.I. Method does not consider quality of material used from pavement

(2) Thickness is suggested same for poor or good quality material.

Example 7

A Soil subgrade sample collected from the site was analysed and the results obtained are as given below:

(i) Soil portion passing 0.074 mm sieve, percent = 60%

(ii) Liquid Limit, percent = 45%

(iii) Plastic Limit, percent = 23%

Determine the group index of subgrade, and design pavement thickness using group index method.

Using following Data :

G.I. Values	Total Thickness Required (cm)
0	22
5	35
10	43
15	48
20	52

Sol. $p = 60\%$; $W_L = 45\%$; $W_P = 23\%$

Plasticity Index $(I_P) = W_L - W_P = 45 - 23 = 22\%$

$$\begin{array}{l|l|l} a = p - 35 & \not> 40 & = 60 - 35 = 25 \\ b = p - 15 & \not> 40 & = 60 - 15 = 45 \\ c = W_L - 40 & \not> 20 & = 45 - 40 = 5 \text{ , take } b = 40 \\ d = I_P - 10 & \not> 20 & = 22 - 10 = 12 \end{array}$$

$G.I. = 0.2a + 0.005\ ac + 0.01\ bd$

$= (0.2 \times 25) + (0.005 \times 25 \times 5) + 0.01\ (40)\ (12) = 10.425$

So, thickness required

$$= 43 + \frac{48 - 43}{5} \times (10.425 - 10) \quad \text{[By Interpolation]}$$

$$= 43.425 = 43.50\,\text{cm}$$

CALIFORNIA BEARING RATIO METHOD

- CBR method of flexible pavement design is based on strength parameter of subgrade soil and subsequent pavement material.
- In order to design a pavement by CBR method, first the soaked CBR value of the soil subgrade is evaluated and the appropriate design curve is chosen by taking the design wheel load as given in figure or by taking the anticipated traffic into consideration.
- The total thickness of pavement needed to cover the subgrade of the known CBR value is obtained
- In case there is a material superior than the soil sub-grade, such that it may be used as sub base course then the thickness of construction over this material could be obtained from the design chart knowing the CBR value of the sub-base.
- Thickness of sub-base is the total thickness minus the thickness over the sub-base.
- Based on CBR value of any material, over which a flexible pavement is required, thickness of pavement over this is given by

$$t = \sqrt{P}\left[\frac{1.75}{CBR} - \frac{1}{p\pi}\right]^{1/2}$$

$$= \left[\frac{1.75\,P}{CBR} - \frac{P}{p\pi}\right]^{1/2}$$

$$t = \left[\frac{1.75P}{CBR} - \frac{A}{\pi}\right]^{1/2} \quad \text{... (i)}$$

Note: This formula is applicable only when the CBR value of the subgrade soil is less than 12%.

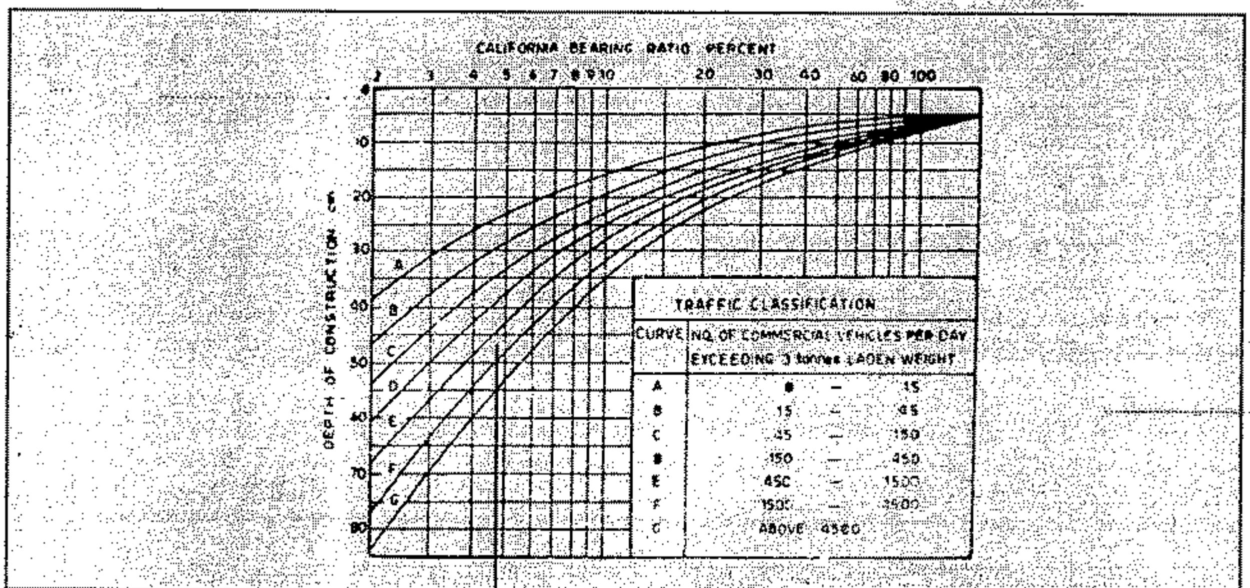
Here, t = pavement thickness, cm

P = wheel load, kg

CBR = California Bearing Ratio, percent

p = tyre pressure, kg/cm^2

A = area of contact, cm^2 [if radius of contact area is r, $A = \pi r^2$]



C.B.R. Design Chart [Recommended by IRC as per IRC37 – 1970]

IRC RECOMMENDATIONS [AS PER IRC 37:1970]

1. CBR test should be performed on remoulded soils in the laboratory, insitu test are not recommended for design purpose.
2. The soil should be compacted at OMC to proctor density.
3. Test samples should be soaked in water for 4 days period before testing. However in dry zone (< 50 cm rainfall) it is not necessary to soak.
4. At least 3 samples should be tested on each type of soil at the same density and moisture content, if variation is more than permissible value, an average of six samples should be considered.

Permissible variations	CBR(%)
3%	up to 10%
5%	10 to 30%
10%	30 to 605

5. The top 50 cm of subgrade should be compacted at least up to 95 to 100% of proctor density
6. Following formula may be used in case estimating future heavy vehicles in view of growth rate for design

$$A = P(1+r)^{n+10}$$

Where, A = number of heavy vehicles/day for design (weight > 3 tonnes).
P = No. of heavy vehicles per day at last count.
r = annual rate of increase of vehicles
n = No. of years between the last count and the year of completion of construction.

P should be seven days average and, r can be taken as 7.5 % for roads in rural areas.

7. The design thickness is considered applicable for single axle loads upto 8200 kg and tendem axle loads up to 14500 kg. For higher axle loads, the thickens values should be further increases.
8. When sub base course materials contain substantial proportion of aggregates of size above 20 mm, the CBR value of materials would not be valid for the design of subsequent layer above them.

Limitations

The CBR method gives the total thickness requirement of the pavement above a subgrade and this thickness requirement of the pavement above a subgrade and this thickness value would remain same irrespective of the quality of materials used in component layers. Thus the component of materials should be judiciously chosen for durability and economy.

Example 8

Soil subgrade sample was obtained from the project site and the CBR test was conducted at field density. The following were the results.

Penetration mm	Load kg	Penetration mm	Load kg
0.0	0.0	3.0	56.5
0.5	5.0	4.0	67.5
1.0	16.2	5.0	75.2
1.5	28.1	7.5	89.0
2.0	40.0	10.0	99.5
2.5	48.5	12.5	106.5

It is desired to use the following material for different pavement layers.

(i) Compacted sandy soil with 7 percent CBR
(ii) Poorly graded gravel with 20 percent CBR
(iii) Well graded gravel with 95 percent CBR
(iv) Minimum thickness of bituminous concrete surfacing may be taken as 5 cm

The traffic survey revealed the present ADT of commercial vehicle as 1200. The annual rate of growth of traffic is found to be 8 percent. The pavement construction is to be completed in three years after the last traffic count.

(a) Design the pavement section by CBR method as recommended by IRC (37:1970), using all the four pavement materials,
(b) Suggest alternate design without using poorly graded gravel.

Sol.

CBR value of Soil Subgrade

The plot is made between load in kg versus penetration of plunger for the test data obtained for soil subgrade as given in figure. Loads at 2.5 and 5.0 mm penetration (after correction) are 55 and 78 kg respectively.

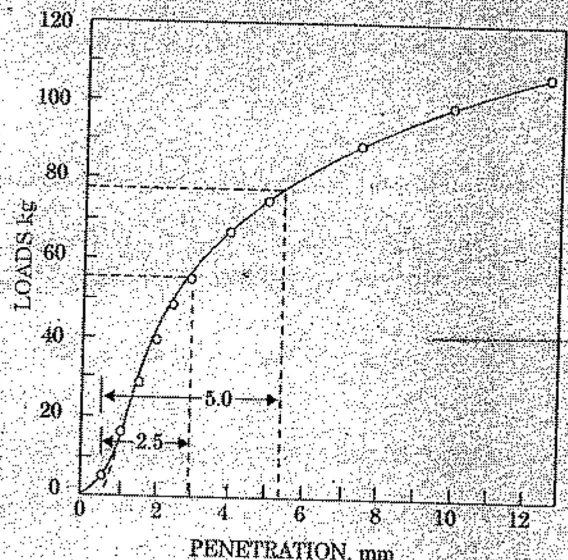
Area of plunger of dia 5 cm = 19.6 cm^2.

Pressure at 2.5 mm penetration = $\frac{55}{19.6}$ kg / cm^2.

Pressure at 5 mm penetration = $\frac{78}{19.6}$ kg / cm^2

$$\text{C.B.R. value of soil at 2.5 mm} = \frac{\text{Pressure on plunger @ 2.5 mm penetration for soil}}{\text{Pressure as above for standard crushed stones}} \times 100$$

$$= \frac{55}{19.6} \times \frac{100}{70} = 4.0 \text{ percent}$$



Load-Penetration Curve

$$\text{CBR of Soil at 5 mm} = \frac{78 \times 100}{19.6 \times 105} = 3.8 \text{ percent}$$

Adopt CBR value = 4.0 percent

Calculation of Design Thickness of Different Layers

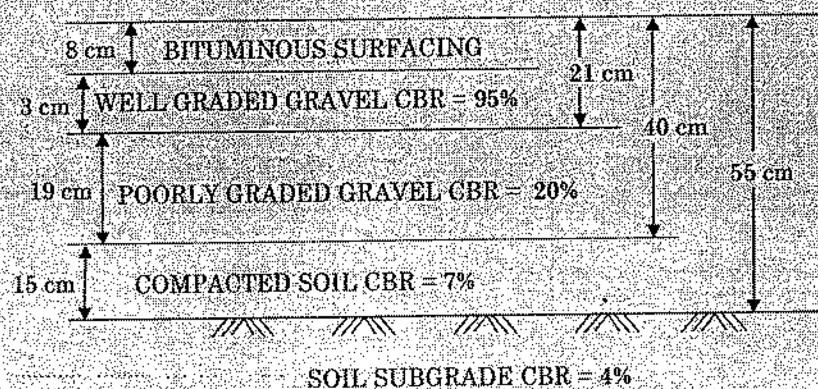
No. of vehicles for design is given by

$$A = P(1+r)^{(n+10)} = 1200\left[1 + \frac{8}{100}\right]^{(3+10)} = 3260 \text{ vehicles/day}$$

Therfore Design Curve F is to be used for design as the design traffic volume is in the range 1500 to 4500 cv/day.

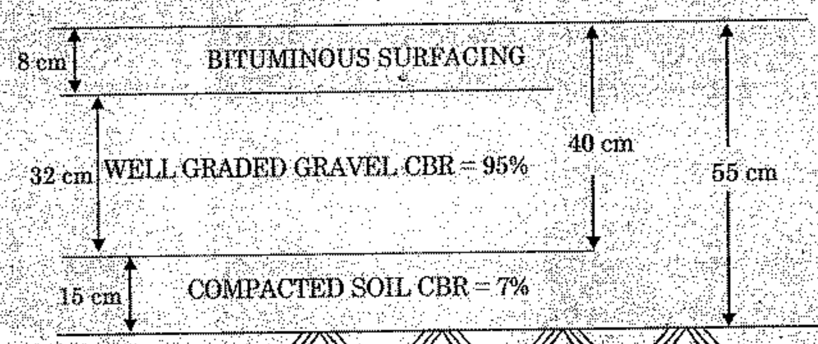
- Using the design chart vide figure, the total pavement thickness over subgrade having CBR of 4 percent is obtained as 55 cm for curve F.
- Thus 55 cm of pavement materials is required to cover the natural soil subgrade having 4% CBR value. Now to compute the thickness of compacted soil, the design curve F is again used for CBR value of 7 per cent.

- Pavement thickness of 40 cm is required above the compacted soil subgrade having CBR value of 7 per cent and hence the actual thickness of this layer is 55 – 400 = 15 cm.
- Similarly the thickness of pavement required over poorly graded gravel of CBR 20 percent and well graded gravel of CBR 95 per cent are 21 cm and 8 cm respectively.



Pavement Section by CBR Method

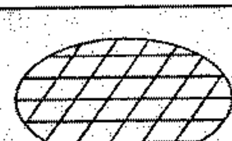
Alternativley, if it is considered not to use poorly graded gravel as employed above, then the design section would be as shown in figure



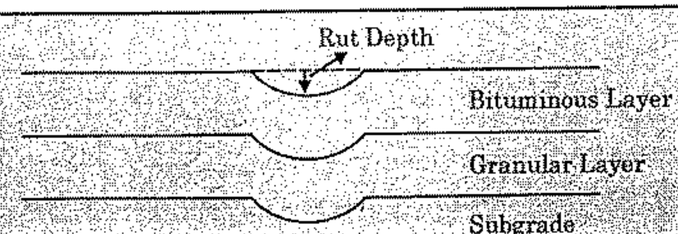
SOIL SUBGRADE CBR = 4% **(Alternate Pavement Section)**

Design criteria for flexible pavement design [As per IRC 37:200]

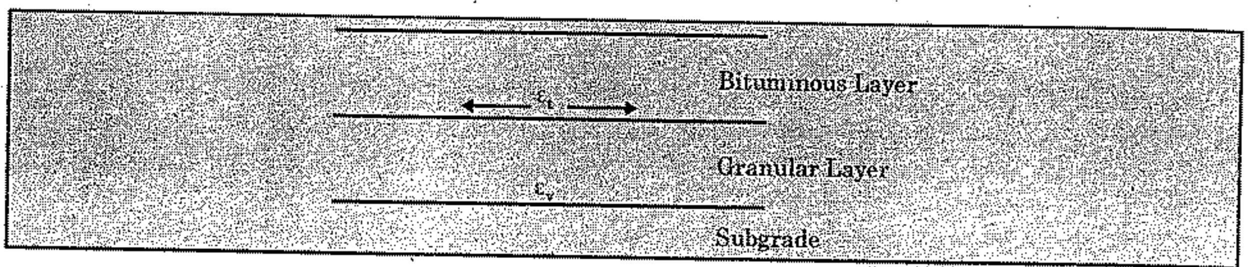
- The pavement should be designed such that fatigue cracking of bituminous layer is not excessive and also rutting due to permanent deformation in subgrade should not be excessive.
- Fatigue cracking in bituminous layer looks like crocodile cracking.



- Rutting is the permanent deformation caused mainly due to permanent deformation in subgrade.



- Causative factor for permanent deformation of subgrads is the vertical strain (ε_v) the subgrade top.
- Causative factor for tensile cracking due to fatigue is the tensile strain (ε_t) at the bottom of bitumenous layer.



- Thus in the design of flexible pavement, ε_v and ε_t are to be controlled.
- Acceptable criteria for fatigue cracking is that during design life not more than 20% of the paved area should have fatigue cracking.
- Pavement layer thickness and material should be selected such that computed strains are within acceptable limit corresponding to the design traffic selected.
- Emperical relations are available which relate the cumulative standard axle lood repetitions before pavement develops 20 mm ruting depth with initial vertical strain (ε_v) soon after construction and also for cumulative standard axle load before pavement develops 20% fatigue cracking with tensile strain (ε_t) and classic modulus of asphatic concrete.

Rutting Criteria :

$$N_R = 4.1656 \times 10^{-8}\left(\frac{1}{\varepsilon_v}\right)^{4.5337} \qquad \text{.........(A)}$$

N_R = No. of cumulative standard axles to produce rutting of 20 mm.

ε_v = Vertical subgrade strain

Fatigue Criteria :

$$N_F = 2.21 \times 10^{-4}\left(\frac{1}{\varepsilon_t}\right)^{3.89}\left(\frac{1}{E}\right)^{0.854} \qquad \text{...(B)}$$

N_F = No. of cumulative standard axle to produces 20% cracked surface area.

ε_t = Tensile strain at the bottom of BC layer

E = Elastic modulus of bituminous surfacing.

Cumulative no. of standard axle for the design life of the pavement is calculated from the formula as given below :

$$N = \frac{365A * D * F\left((1+r)^n - 1\right)}{r}$$

N = No. of standard axles to be catered for in design.

A = Initial traffic in the year of completion of construction in terms of no. of commercial vehicle/day.

D = Lane distribution factor

F = Vehicle damage factor

n = Desing life in years

r = Annual rate of growth of commercial vehicles (i.e. for 7.5% growth rate take r = 0.075)

$A = P(1+r)^x$

P = No. of commercial vehicles as per last count

x = No. of years between the last count and the year of completion of construction.

Thus for design traffic N, ε_v and ε_t are calculated from relation A and B as described before.

Thus pavement thickness and material should be selected such that initial stress should be less than that obtained from A and B for the given design traffic (N).

IRC also gives design chart for thickness of various pavement layers depending upon the CBR value of subgrade (Range 2 to 10%) and design traffic in million standard axles (range 1 - 150 msa)

Traffic Growth Rate

Traffic growth rates should be estimated

- by studying the past trends of traffic growth and
- by establishing econometric models, as per the procedure outlined in IRC ; 108 "Guidelines for Traffic Prediction on Rural Highways".
- If adequate data is not available, it is recommended that an averae annual growth rate of 7.5 per cent may be adopted.

Design Life

- For the design of pavement, the design life is defiend 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 National Highways and State Highways should be designed for a life of 15 years.
- Expresways and urban roads may be designed for a longer life of 20 years.
- For other categories of roads, a design life of 10 to 15 years may be adopted.

Vehicle Damage Factor :

- The vehicle damage factor (VDF) is a amultiplier to convert the number of commercial vehcles of different axle loads and axle configuration 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 vehicle axle configuration, axle loading, terrain, type of road and from region to region.
- The VDF is arrived at from axle load surveys on typical road sections so as to cover various influencing factors, such as traffic mix, mode of transportation commodities carried, time of the year, terrain, road conditions and degree of enforcement.
- The equivalent axle load factor for any axle load is given by

  a) Single axle load

$$EALF = \left(\frac{\text{axle load in kg}}{8160}\right)$$

b) For Tandem axle load

$$EALF = \left(\frac{\text{axle load in kg}}{14968}\right)$$

- Where sufficient information on axle load is not available and the project size does not warrant conducting an axle load survey, the indicative value of vehicle damage factor as given in the following table may be used.

initial traffic volume in terms of number of commercial vehicle per day	Terrain	
	Rolling Plain	Hilly
0-150	1.5	0.5
150-1500	3.5	1.5
> 1500	4.5	2.5

DISTRIBUTION OF COMMERCIAL TRAFFIC OVER THE CARRIAGE WAY

- A realistic assessment of distribution of commercial traffic by direction and by lane is necessary as it directly affects the total equialent standard axle load applications used in the design.
- In the absence of adequate and conclusive data for Indian conditons, it is recommended that for the time being the following distribution may be assumed for design until more reliable data on placement of commercial vehicles on the carriageway lanes are available :

  i) Single-lane roads

  Traffic tends to be more channelised on single-lane 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.

  ii) Two-lane single carriageway roads

  The design should be based on 75 per cent of the total number of commercial vehicle in both directions.

  iii) Four-lane single carriageway roads

  The design should be based on 40 per cent of the total number of commercial vehicles in both directions.

  iv) Dual carriageway roads

  - The design of dual two-lane carriageway roads should be based on 75 per cent 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 per cent and 45 per cent respectively.

- The traffic in each direction may be assumed to be half of the sum in both directions when the latter only is known. Where significant difference between the two streams can occur, condition in the more heavily trafficked lane should be considered for design.

Example 9

A two-lane two-way road is at present carrying a traffic of 1000 commercial vehicles per day. It is to be strengthened for the growing traffic needs. The vehicle damage factor has been found to be 3.0. The rate of growth of traffic is 10 per cent per annum. The period of construction is 5 years. The pavement is to be designed for 15 years after completion. Calculate the cumulative standard axles to be used in design.

Sol. Present traffic = 1000 cv/day

Traffic after completion of strengthening = 1000 $(1 + 0.1)^5$ = 1611 cv/day

Number of commercial vehicles per day in design lane = 1611 × 0.75

$$\text{Cummulative standard axles} = \frac{365 \times 1611 \times 0.75 \times 3\left[(1+0.1)^{15}-1\right]}{0.1}$$

$$= \frac{365 \times 1611 \times 0.75 \times 3 \times 3.177}{0.1 \times 10^6}$$

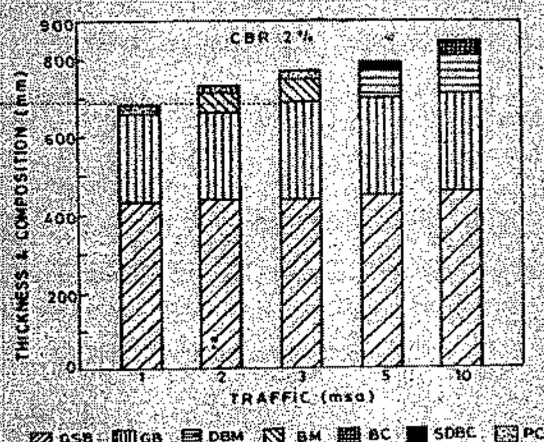
= 42.03 million standard axles (msa).

RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

PAVEMENT DESIGN CATALOGUE

PLATE 1 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

CBR 2%					
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION			
		Bituminous Surfacing		Granular Base (mm)	Granular Sub-base (mm)
		Wearing Course (mm)	Binder Course (mm)		
1	660	20 PC		225	435
2	715	20 PC	50 BM	225	440
3	750	20 PC	60 BM	250	440
5	795	25 SDBC	70 DBM	250	450
10	850	40 BC	100 DBM	250	460

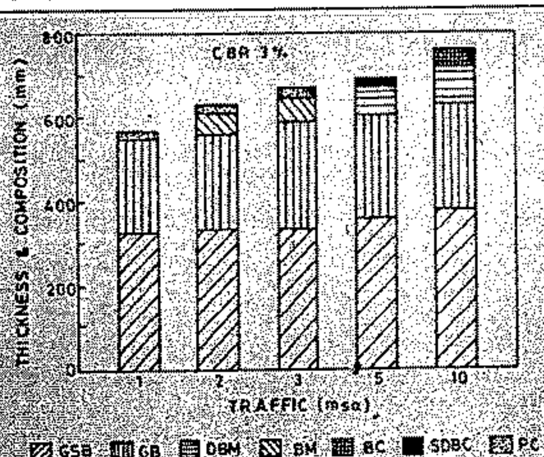


RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

PAVEMENT DESIGN CATALOGUE

PLATE 1 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

CBR 3%					
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION			
		Bituminous Surfacing		Granular Base (mm)	Granular Sub-base (mm)
		Wearing Course (mm)	Binder Course (mm)		
1	550	20 PC		225	435
2	610	20 PC	50 BM	225	335
3	645	20 PC	60 BM	250	335
5	690	25 SDBC	60 DBM	250	335
10	760	40 BC	90 DBM	250	380

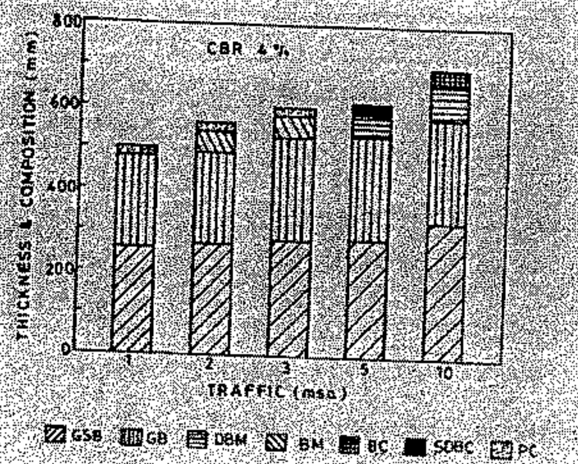


RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

PAVEMENT DESIGN CATALOGUE

PLATE 1 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

CBR 4%					
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION			
		Bituminous Surfacing		Granular Base (mm)	Granular Sub-base (mm)
		Wearing Course (mm)	Binder Course (mm)		
1	480	20 PC		225	255
2	540	20 PC	50 BM	225	265
3	580	20 PC	50 BM	250	280
5	620	25 SDBC	60 DBM	250	285
10	700	40 BC	80 DBM	250	330

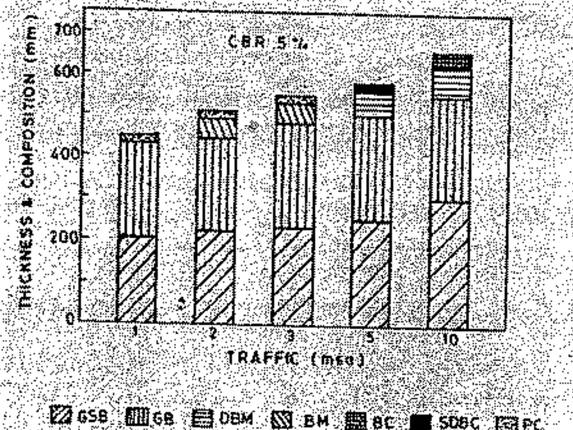


RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

PAVEMENT DESIGN CATALOGUE

PLATE 1 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

CBR 5%					
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION			
		Bituminous Surfacing		Granular Base (mm)	Granular Sub-base (mm)
		Wearing Course (mm)	Binder Course (mm)		
1	430	20 PC		225	205
2	490	20 PC	50 BM	225	215
3	530	20 PC	50 BM	250	230
5	580	25 SDBC	55 DBM	250	250
10	660	40 BC	70 DBM	250	300

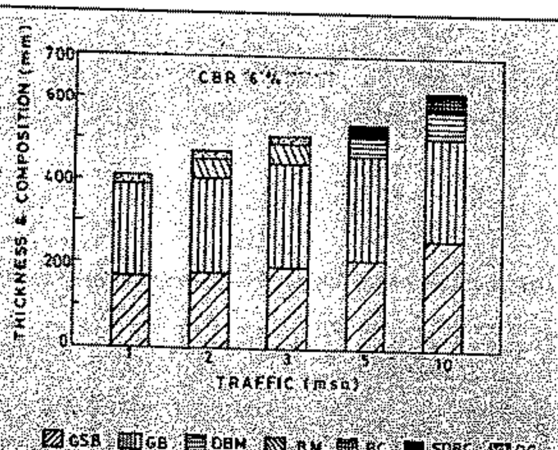


RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

PAVEMENT DESIGN CATALOGUE

PLATE 1 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

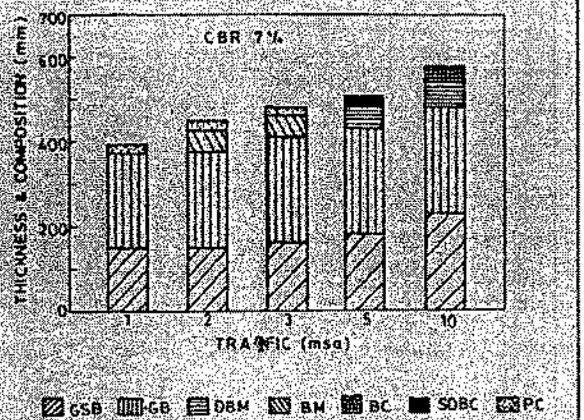
CBR 6%					
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION			
		Bituminous Surfacing		Granular Base (mm)	Granular Sub-base (mm)
		Wearing Course (mm)	Binder Course (mm)		
1	390	20 PC		225	165
2	450	20 PC	50 BM	225	175
3	490	20 PC	50 BM	250	190
5	535	25 SDBC	50 DBM	250	210
10	615	40 BC	65 DBM	250	260



RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

PAVEMENT DESIGN CATALOGUE
PLATE 1 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

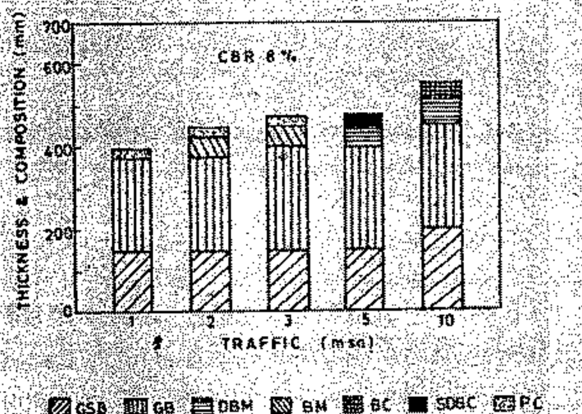
CBR 7%					
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION			
		Bituminous Surfacing		Granular Base (mm)	Granular Sub-base (mm)
		Wearing Course (mm)	Binder Course (mm)		
1	375	20 PC		225	150
2	425	20 PC	50 BM	225	150
3	460	20 PC	50 BM	250	160
5	505	25 SDBC	50 DBM	250	180
10	580	40 BC	60 DBM	250	230



RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

PAVEMENT DESIGN CATALOGUE
PLATE 1 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

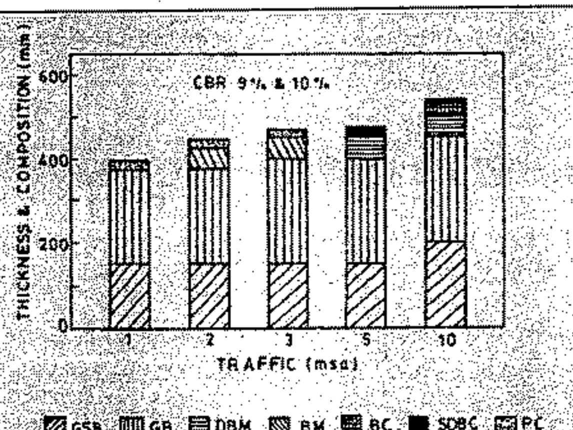
CBR 8%					
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION			
		Bituminous Surfacing		Granular Base (mm)	Granular Sub-base (mm)
		Wearing Course (mm)	Binder Course (mm)		
1	375	20 PC		225	150
2	425	20 PC	50 BM	225	150
3	450	20 PC	50 BM	250	150
5	475	25 SDBC	50 DBM	250	150
10	550	40 BC	60 DBM	250	200



RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

PAVEMENT DESIGN CATALOGUE
PLATE 1 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

CBR 9% & 10%					
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION			
		Bituminous Surfacing		Granular Base (mm)	Granular Sub-base (mm)
		Wearing Course (mm)	Binder Course (mm)		
1	375	20 PC		225	150
2	425	20 PC	50 BM	225	150
3	450	20 PC	50 BM	250	150
5	475	25 SDBC	50 DBM	250	150
10	540	40 BC	50 DBM	250	200

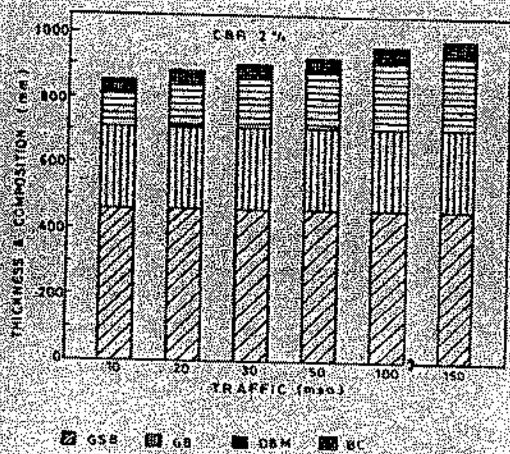


RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-150 msa

PAVEMENT DESIGN CATALOGUE

PLATE 2 - RECOMMENDED DESIGNS FOR TRAFFIC RANGE 10-150 msa

CBR 2%				
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION		
		Bituminous Surfacing		Granular Base & Sub-base (mm)
		BC (mm)	DBM (mm)	
10	850	40	100	Base = 250
20	880	40	130	
30	900	40	150	
50	925	40	175	Sub-base = 460
100	955	50	195	
150	975	50	215	



Design Data as per IRC 37:2012

- Only the number of commercial vehicles having gross vehicle weight of 30 kN or more and their axle-loading is considered for the purpose of design of pavement
- Assessment of the present day average traffic should be based on seven-day-24-hour count made in accordance with IRC : 9-1972 "Tarffic Census on Non-Urban Roads

Traffic Growth Rate

- If the data for the annual growth rate of comercial vehicles is not available or if it is less than 5 per cent, a growth rate of 5 per cent should be used (IRC : SP : 84-2009).
- It is recommended that pavements for National Highways and state Highways should be designed for a minimum life of 15 years. Expresways and Urban Roads may be designed for a longer life of 20 years or higher using innovative design adopting high fatigue bituminous mixes.
- For other categories of roads, a design life of 10 to 15 years may be adopted.
- The equations for computing equivalency factors for single, tandem and tridem axles given below should be used for converting different axle load repetitions into equivalent standard axle load repetitions.

Single axle with single wheel on either side $= \left(\frac{\text{axle load in kN}}{65}\right)^4$

Single axle with dual wheels on either side $= \left(\frac{\text{axle load in kN}}{80}\right)^4$

Tandem axle with dual wheels on either side$= \left(\frac{\text{axle load in kN}}{148}\right)^4$

Tridem axle with dual wheels on either side $= \left(\frac{\text{axle load in kN}}{224}\right)^4$

- VDF should be arrived at carefully be carrying out specific axle load surveys on the existing roads

- Minimum sample size for survey is given in Table below.

Total number of commercial vehicles per day	Minimum percentage of commercial Tarffic to be surveyed
<3000	20 per cent
3000 to 6000	15 per cent
>6000	10 per cent

- Where sufficient information on alxe loads is not available and the small size of the project does not warrant an axle load survey, the default values of vehicle damage factor as given in Table below may be used.

Inital traffic volume in terms of commercial vehicles per day	Terrain	
	Rolling / Plain	Hilly
0 - 150	1.5	0.5
150 - 1500	3.5	1.5
More than 1500	4.5	2.5

Distribution of Commercial Traffic over the Carriageway

i) Single-lane roads

- Traffic tends to be more channelized on single-lane 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 on both directions.

ii) Two-lane single carriagewayroads

- The design should be based on 50 percent of the total number of comercial vehicles in both directions. If vehicle damage factorin one direction is higher, the traffic in the direction of higher VDF is recommended for design.

iii) Four-lane single carriageway roads

The design should be based on 40 per cent of the total number of commercial vehicles in both directions.

iv) Dual carriageway roads

The design of dual two-lane carriageway roads should be based on 75 per cent of the commercial vehicles in each direction. For dual three lane carriageway and dual four-lane carriageway, the distribution factor will be 60 per cent and 45 per cent respectively.

- Similar to IRC 37 : 2001, design charts are available in IRC 37 : 2012 for CBR 3% to 15% and traffic 1-150 MSa.

Note : The formula relating rutting criteria and fatigue criteria in IRC 37:1012 is alsmost similar to that in (IRC 37: 2001) except that the aceptibility criteria has been changed.

For fatigue cracking, cracking in 20% of area has been considered for traffic upto 30 MSa and 10% per traffic beyond 30 MSa.

For rutting, the limiting value is recommended as 20 mm in 20% of length upto 30 MSa and in 10% of length for traffic beyond 30 msa.

Pavement Composition

Sub Grade :

- The subgrade whether in cut of fill should be well compacted to utilise its full strength and to economise thereby on the overall thickness of pavement required.
- For Expressway, National Highways, State Highways and Major District Roads, heavy compaction is recommended.
- The current specification for Road and Bridge Works recommend that the subgrade shall be compacted to 97 per cent of dry density achieved with heavy compaction (modified proctor density).
- This density requirement is recommended for subgrade compaction for Exprssways, National Highways, State Highways, Major District Roads and other heavily trafficked roads.
- In other cases the subgrade should be compacted to at least 97 per cent of the standard proctor density.
- For high category roads like Expressways, National Highways and State Highways the material used for subgrade construction, should have the dry density of not less than 1.75 gm/cc.
- For design, the subgrade strength is assessed in terms of the CBR of the subgrade soil in both fill and cut sections at the most critical mositure conditions likely to occur in-situ.
- Perferably, the subgrade soil should have a CBR value of 2%. Where CBR is less than 2%, the design should be based on subgrade CBR value of 2% and a caping layer of 150 mm thickness of material with a minimum CBR of 10% shall be provided in addition to subbase.

Sub-base

- Sub-base materials comprise natural sand, gravel, laterite, brick, crushed stone or combinations thereof meeting the prescribed grading and physical requirements.
- The sub-base material should have a minimum CBR of 20% and 30% for traffic upto 2 msa and traffic exceeding 2 msa respectively.
- Sub-base usually consist of granular sub base material or of WBM and the thickness should not be less than 150 mm for design traffic less than 10 msa and 200 mm for design traffic of 10 msa and above.

Base

- The recommended design are for unbounded granular base which comprise conventional water bound macadam (WBM) or wet mix macadam (WMM) or equivalent.
- The materials should be of good quality with minimum thickness of 225 mm for traffic upto to 2 msa an 150 mm for traffic exceeding 2 msa.

Bituminous Surfacing

- The surfacing consists of a wearing course or a binder course plus wearing course.
- The most commonly used wearing courses are surface dressing, open graded premix carpet, mix seal surfacing, semi-dense bituminous concrete and bituminous concrete. For binder course, MOST specifies, it is desirable to use bituminous macadam (BM) for traffic upto 5 msa and dense bituminous macadam (DBM) for traffic more than 5 msa.

Example 10

Design the pavement for construction of a new bypass with the following data: (using, IRC 37: 2001)

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%.

Sol.

1. Distribution factor = 0.75
2. $N = \frac{365 \times [(1+0.075)^{15} - 1)]}{0.075} \times 400 \times 0.75 \times 2.5 = 7200000 = 7.2$ msa
3. Total pavement thickness for CBR 4% and traffic 7.2 msa from IRC: 37 2001 chart 1 = 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%

Example 11

Design the pavement for construction of a new two lane carriage for design life 15 years using IRC method. The initial traffic in the year of completion in each direction is 150 CVPD and growth rate is 5%. Vehicle damage factor based on axle load survey = 2.5 std axle per commercial vehicle. Design CBR of subgrade soil = 4%.

Sol.

1. Distribution factor = 0.75
2. $N = \frac{365 \times [(1+0.05-1)]}{0.05} \times 300 \times 0.75 \times 2.5 = 4430348.837 = 4.4$ msa
3. Total pavement thickness for CBR 4% and traffic 4.4 msa from IRC:37 2001 chart 1 = 580 mm
4. Pavement composition can be obtained by interpolation from Pavement Design Catalogue (IRC: 37 2001)
   (a) Bituminous surfacing = 20 mm PC + 50 mm BM
   (b) Road-base = 250 mm Granular base
   (c) sub-base = 280 mm granular material.

CALIFORNIA RESISTANCE VALVE METHOD

- This method is based on
  (a) Stabilometer R-value
  (b) Cohesinometer C-value
- Thickness of the pavement is given by

$$t\ (cm) = \frac{K(TI)(90-R)}{C^{1/5}}$$

K = Numerical constant = 0.166

t = thickness of pavement in cm.

TI (Traffic Index) = $1.35(EWL)^{0.11}$

EWL = Equivalent wheel load

R = Stabilometer value

C = Cohesinometer value

- The value of EWL is obtained from the average annual daily traffic of particular number of axle and EWL constant.

No. of axle	EWL constant
2	330
3	1070
4	2460
5	4620

Example : If the traffic survey gives data as follows.

S.No.	No. of axle	AADT
1	2	3750
2	3	470
3	4	320
4	5	120

$$EWL = \sum_{1}^{4}(EWL\,constant)\times(AADT)$$

- This value of EWL will give the annual value of equivalent wheel load.

$$EWL = (330 \times 3750) + (1070 \times 470) + (2460 \times 320) + (462 \times 120)$$
$$= 3082000$$

- The cohesionometer meter value "C" is obtained for each layer of pavement material separately from the test and composite or equivalent "C" value for the complete pavement will be estimated if the thickness of each layer and "C" value of that layer is known.
- t_1 thickness of material with C value equal to C_1 is equivalent to t_2 thickness of another material with C value equation C_2. The relation is given as

$$\frac{t_1}{t_2} = \left(\frac{C_2}{C_1}\right)^{1/5}$$

- While designing a pavement as the thickness of pavement is not known in advance, it is easier if pavement is first assumed to be consisting of one material only. For known C value subsequently, the individual thickness of each layer will be calculated from equivalency relationship.

$$\frac{t_1}{t_2} = \left(\frac{C_2}{C_1}\right)^{1/5}$$

Example 12

Calculate the equivalent C-value of a three layer-pavement having individual material as shown below.

Material	Thickness	C-value
(1) Bituminous layer	12.5 cm	62
(2) Cement treated base	25.0 cm	180
(3) Well graded gravel	20.0 cm	25

Sol. Let us convert all thickness in terms of cement base

(1) *For bitminous layer*

$$T_B = 12.5 \text{ cm} \; ; \quad T_C = ? \; ; \; C_B = 62 \; ; \; C_C = 180$$

$$\frac{T_B}{T_C} = \left(\frac{C_C}{C_B}\right)^{1/5}$$

$$\frac{12.5}{T_C} = \left(\frac{180}{62}\right)^{1/5}$$

$$T_C = 10.10 \text{ cm}$$

(2) *For cement treated base*

$$T_C = 25 \text{ cm}$$

(3) *Well graded gravel*

$$T_g = 20.0 \text{ cm} \; ; \; T_C = ? \; ; \; C_g = 25 \; ; \; C_C = 180$$

$$\frac{T_g}{T_C} = \left(\frac{C_C}{C_g}\right)^{1/5}$$

$$\frac{20}{T_C} = \left(\frac{180}{25}\right)^{1/5}$$

$$T_C = 13.47 \text{ cm}$$

Interms of cement treated base.

$\therefore$ Total thickness of pavment = 10+25 +13.47

$$T_C = 48.57 \text{ cm}$$

$$C_c = 180$$

Thickness of pavment 57.5 = T_P

$$C_P = ?$$

$$\frac{T_P}{T_C} = \left(\frac{C_C}{C_P}\right)^{1/5}$$

$$\frac{57.5}{48.57} = \left(\frac{180}{C_P}\right)^{1/5}$$

$$2.325 = \frac{180}{C_P}$$

$$C_P = 77.40$$

Example 13

Calculate the equivalent C-value of a three layout pavement section having individual C-values as given below :

Materials	**Thickness (cm)**	**C-Value**
Bituminous Concrete	10	60
Cement Treated Bars	20	225
Gravel Sub Base	10	15

Sol. The individual thickness of each layer is converted to their respective gravel equivalent We know that

$$\frac{t_g}{t} = \left(\frac{C}{C_g}\right)^{1/5}$$

$$\therefore \quad \frac{t_g}{10} = \left(\frac{60}{15}\right)^{1/5}$$

$$t_g = 13.2 \text{ cm} \quad \ldots\ldots \quad \text{(i)}$$

For Base Course

$$\frac{t_g}{20} = \left(\frac{C}{C_g}\right)^{1/5}$$

$$t_g = \left(\frac{225}{15}\right)^{1/5} \times 20 = 34.4 \quad \ldots\ldots \quad \text{(ii)}$$

For Sub Base Course $t_g = 10\text{cm}$ ..... (iii)

Therefore Actual pavement thickness

$$= 10 + 20 + 10 = 40 \text{ cm}$$

This is equivalent to gravel thickness

$$= 10 + 13.2 + 34.4$$

$$= 57.6 \text{ cm}$$

Therefore, $\frac{t_g}{T} = \left(\frac{C}{C_g}\right)^{1/5}$

$\Rightarrow$ $\left(\frac{t_g}{T}\right)^5 = \left(\frac{C}{C_g}\right)$

$$C = \left(\frac{t_g}{T}\right)^5 \times C_g = \left(\frac{57.6}{40}\right)^5 \times 15$$

$\therefore$ $C = 92.876 \approx 93$

Example 14

Calculate 10 year EWL values traffic index using following data

No. of axle	2	3	4	5
AADT volume (Avg. annula daily traffic)	3750	470	320	120

Assume 60% increase in traffic in next 10 years period calculate thickness of pavment required using california resistance value method;

R Value 48, C value = 16

Sol.

No. of axle	AADT value	EWL constant	Total yearly EWL value
2	3750	330	1237500
3	470	1070	502900
4	320	2460	787200
5	120	4620	5554400
			Total 3082000

EWL value at present = 3082000

EWL after 10 year period = 1.60 × 3082000 = 4931200

Avg. EWL value

$$= \frac{3082000 + 4931200}{2} = 4006600$$

Traffic index

$$\text{T.I.} = 1.35\ (\text{EWL})^{0.11}$$

$$= 1.35\ (400660)^{0.11} = 7.1885$$

Thk. of pavement required

$$T = \frac{0.166 \times \text{T.I} \times (90.\text{R})}{C^{1/5}}$$

$$= \frac{0.166 \times 7.1885 \times (90-48)}{(16)^{1/5}} = 28.80 \text{ cm}$$

TRIAXIAL METHOD

- The pavement thickness T_s consisting of material with modulus E_s is given by the equation:

$$T_s = \sqrt{\left(\frac{3P\,XY}{2\pi\,E_s\,\Delta}\right)^2 - a^2} \quad \text{... (i)}$$

P = Wheel load in Kg ; Δ = design deflection (0.25 cm)

a = radiusof contact area (cm) ; X = Traffic coefficient

Y = Satuation coefficient ; E_S = Modulus of elasticity in Kg/cm^2

The recommended values of coefficients X and Y based on ADT of design traffic and rainfall are given below:

Traffic coefficient (X)	ADT (number)
1/2	40–400
2/3	401–800
5/6	801–1200
1	1201–1800
7/6	1801–2700
8/6	2701–4000
9/6	4001–6000
10/6	6001–9000
11/6	9000–13,500
12/6	13501–20,000
Rainfall coefficient (Y)	**Average annual rainfall, cm**
0.5	38–50
0.6	51–64
0.7	65–76
0.8	77–90
0.9	91–100
1.0	101–127

- If pavement and subgrade are considered as a two layer system a *stiffness factor* has to be introduced to take into account the different values of modulus of elasticity of the two layers.
- The pavement thickness is then modified using the stiffness factor equal to $(E_s/E_p)^{1/3}$ where E_s and E_p are values of modulus of elasticity of the subgrade and pavement, respectively. Thus the thickness of pavement, T_p is calculated from the relation:

$$T_p = \left\{\sqrt{\left(\frac{3P\,XY}{2\pi\,E_s\,\Delta}\right)^2 - a^2}\right\}\left(\frac{E_s}{E_p}\right)^{1/3} \quad \text{... (ii)}$$

- The relation between pavement layers of thickness t_1 and t_2 of elastic modulus E_1 and E_2 is given by:

$$\frac{t_1}{t_2} = \left(\frac{E_2}{E_1}\right)^{1/3}$$

Example 15

Design a pavement section by Triaxial method, using following data

Wheel load 4050 kg = P

Radius of contact area = 15 cm = a

Traffic coefficient = 1.6 = x

Rainfall coefficient = 0.7 = y

design deflection Δ = 0.25 cm

Modulus of elasticity of soil subgrade E_s = 120 kg/cm²

Modulus of elasticity of base course material – E_B = 360 kg/cm²

A bituminous layer 7 cm is provided at top, with E_{Bit} = 1200 kg/cm²

Sol.

For two layer system (considering Base course Material)

$$T_B = \left\{\sqrt{\left(\frac{3P\,XY}{2\pi\,E_s\,\Delta}\right)^2 - a^2}\right\}\left(\frac{E_s}{E_p}\right)^{1/3} = \left(\sqrt{\left(\frac{3\times4050\times1.6\times0.7}{2\times\pi\times120\times0.25}\right)^2 - (15)^2}\right)\times\left(\frac{120}{360}\right)^{1/3}$$

$$T_B = 48.96 \text{ cm}$$
$$T_{Bt} = 7.0 \text{ cm}$$
$$E_{Bt} = 1200 \text{ kg/cm}^2$$

7 cm of bituminous layer is equivalent to T_B thicknes of base course material.

$$T_B = ?$$
$$E_B = 360 \text{ kg/cm}^2$$

$$\frac{T_{Bt}}{T_B} = \left(\frac{E_B}{E_{Bt}}\right)^{1/3}$$

$$T_B = \left(\frac{E_{Bt}}{E_B}\right)^{1/3}\times T_{Bt} = \left(\frac{1200}{360}\right)^{1/3}\times 7 = 10.45 \text{ cm}$$

$\therefore$ Remaining thikness of base course = 48.96 – 10.45 = 38.50 cm

Mc Leod Method

- From the plate load tests an empirical design equation was recommended:.

$$T = K\log_{10} P/S$$

T = required thickness of gravel base, cm

P = gross wheel load, kg

S = total subgrade support, kg (for the same contact area, deflection and number of repetitions of load P)

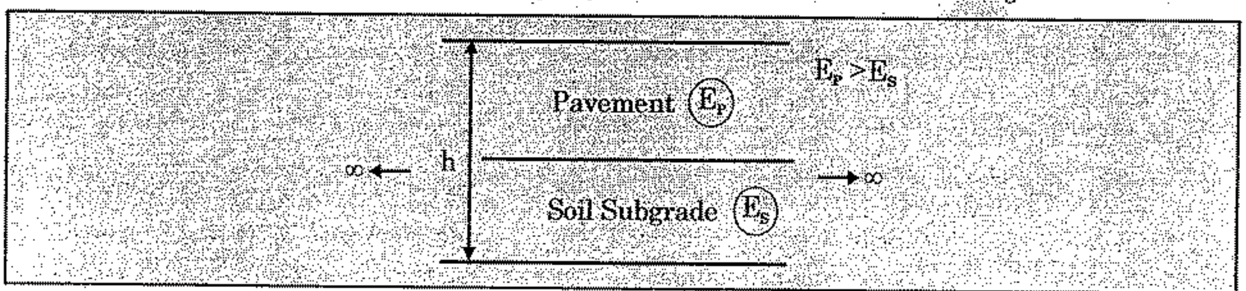
K = base course constant.

- It is found that the base course constant K depends on the loaded area.
- The subgraded support S for the design of highway pavement is calculated from the support measured or calculated for 30 cm diameter plate at 0.5 cm deflection and ten repetitions.

Burmister's (Layered System) Method

Assumption :

1. Materials in each layers are isotropic, homogenous and Elastic.
2. Pavement forms a stiffer layer having higher value of E than that of subgrade.



3. The surface layer is infinite in horizontal directions but finite in vertical direction length and width of infinite, height is finite.
4. Layers are in constant contact.

- Donald M. Burmister developed the layered system analysis. The flexible pavement sections are composed of layers and the elastic modulus of the top layer is the highest. This method is based on Modulus of Elasticity of different layers :

As per Layered System

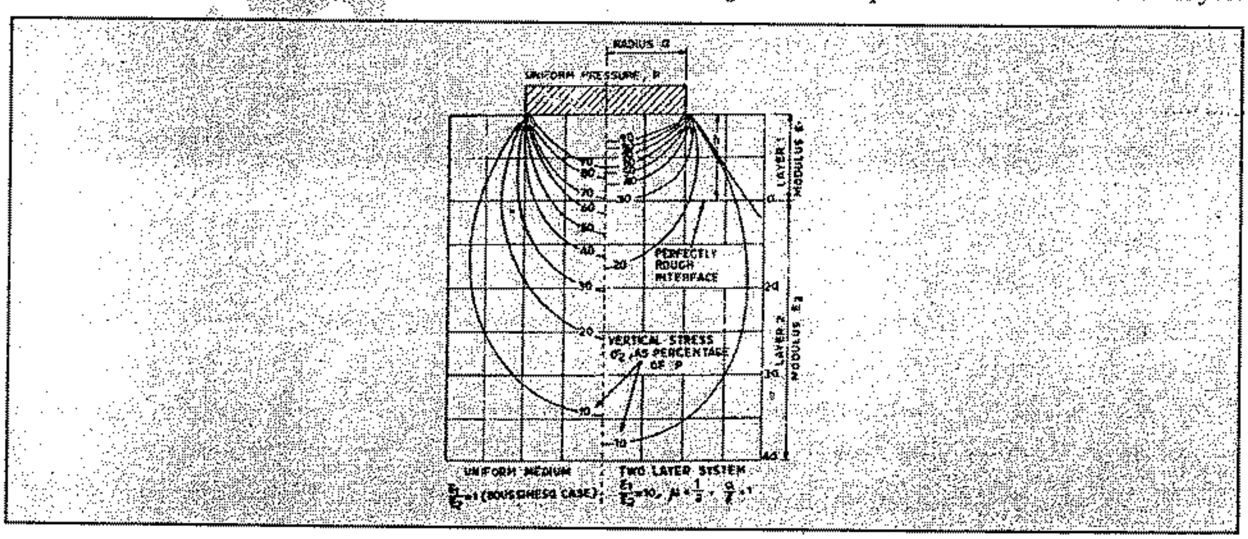
$E_B > E_{SB} > E_S$

(E_B) Base Course

(E_{SB}) Sub-Base Course

(E_{SB}) Soil Subgrade

Young's Modulus of Elasticity of upper layers should be higher as compared to that of lowered layer.



Comparison of Vertical Stress Distribution by Boussinesq and Burmister Approaches.

- It is observed from this figure that the vertical stress on the subgrade is reduced from 70 to 30 percent by introducing a pavement layer of thickness equal to the radius of the load or h = a, having elastic modulus 10 times higher than the elastic modulus of subgrade soil, *i.e.*, for $E_p/E_s = 10$.
- The Burmister's approach therefore utilizes the reinforcing action of the pavement layer.
- The deflection factor F_2 is introduced in two layered system which is dependent on E_s/E_p and h/a.

Displacement Relationship By Burmister's

1. For Flexible Plate.

The wheel load acting over a pavement / on any other surface is considered flexible plate base.

$$\boxed{\Delta = 1.5\frac{pa}{E_s}.F_2} \quad ..... \quad (A)$$

where, Δ = Design deflection.

p = contact pressure at road surface due to wheel load.

a = Radius of contact Area.

E_s = Modulus of Elasticity of Soil Subgrade.

h = Thickness of reinforcing layer.

F_2 = Value as per value of $\frac{E_s}{E_p}$ & $\frac{h}{a}$. This is called deflection factor.

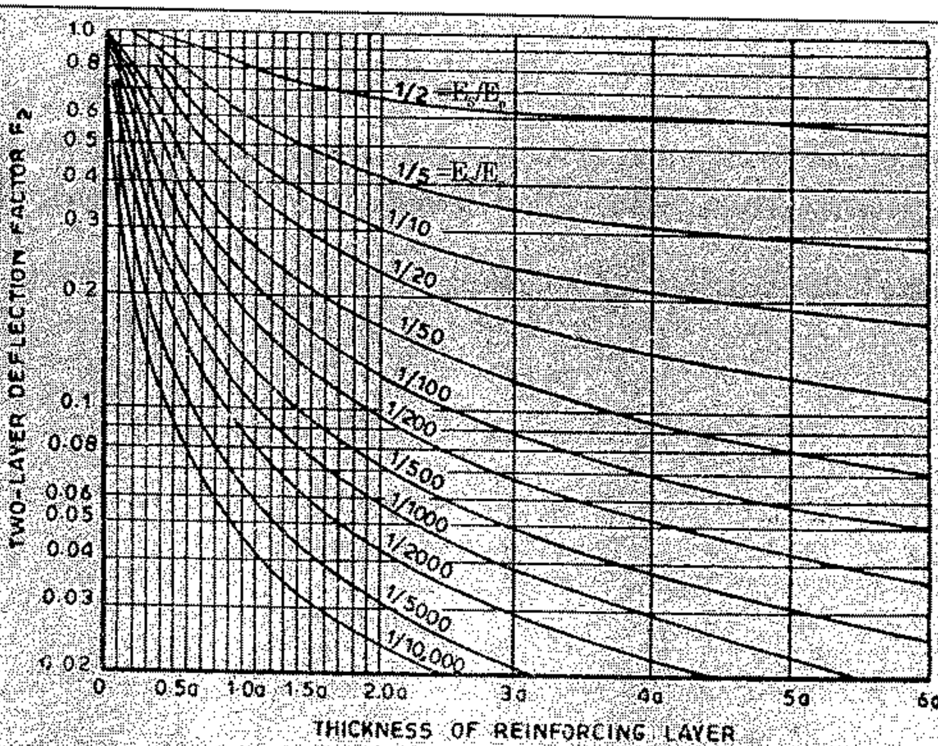
2. For rigid plate

$$\boxed{\Delta = 1.18\frac{pa}{E_s}.F_2}$$

The above formula is applicable, when Rigid plate used. (like steel plate used in plate load test). If plate load test is conducted on soil subgrade only, in this case, thickness of pavement h = 0, that is called single layer system. For which $F_2 = 1.0$

- In the derivation of displacement equations the Poisson's ratio μ is taken as 0.5 both for subgrade and pavement material.

  i.e., $\mu_s = \mu_p = 0.5$
- The plate diameter for load tests may be taken as 30 cm and design deflection may be taken as 0.5 or 0.25 cm.
- It has been recommended that nine trial sections be constructed three each on fill, cut and level areas. In each typical locality, three pavement thickness values equal to $\frac{2}{3}$h, h and 1.5 h are adopted and the actual pavement thickness required for the critical deflection is found.



Relationship of F2 and h in a Two-layer System (Burmister's Method)

Example 16

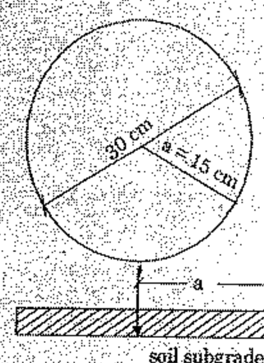
Plate bearing test conducted with 30 cm dia. plate on a soil subgrade yielded a pressure of 1 kg/cm² at 5 mm deflection. A test is carried out over 18 cm base course, yielded a pressure of 5 kg/cm² at 5 mm deflection. Design the section of pavement for wheel load 4100 kg with a tyre pressure of 6 kg/cm² and allowable deflection of 5 mm. use Burmister's method.

Sol.

(I) Plate load test: conducted over soil subgraded.

dia = 30 cm;

∴ a = 15 cm



$$\text{Pressure } p = 1 \text{ kg/cm}^2$$

$$\text{deflectio, } \Delta = 5 \text{ mm} = 0.5 \text{ cm}$$

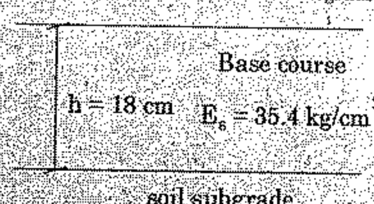
It's a single layer system, $F_2 = 1.0$

∴ Using Rigid plate Burmister formula.

$$\Delta = 1.18 \times \frac{P.a}{E_s} \times F_2$$

$$0.5 = 1.18 \times \frac{1 \times 15}{E_s} \times 1$$

$$E_s = 35.4 \text{ kg/cm}^2$$



(II) Plate load test : Conducted over Base course.

$$\text{Thick. of base course } h = 18 \text{ cm}$$

$$E_s = 35.4 \text{ kg/cm}^2$$

Pressure

$$P = 5\ kg/cm^2\ ;\ \Delta = 5\ mm = 0.5\ cm$$

Using Rigid Plate formula,

$$\Delta = 1.18\frac{P.a}{E_s}\times F_2$$

$$0.05 = 1.18\times\frac{5\times 15}{35.4}\times F_2$$

$$F_2 = 0.2\ mm$$

∴ By using chart. (Given in exam)

$$\frac{h}{a} = \frac{18}{15} = 1.2$$

$\left(\frac{E_s}{E_p}\right)$ value is lying between $\frac{1}{50}$ to $\frac{1}{100}$

$$\therefore \frac{E_s}{E_p} = \frac{1}{80}$$

$$\therefore E_p = 80\times 35.4 = 80\times 35.4$$

$$E_p = 2832\ kg/cm^2$$

(III) Design of pavement

$$P = 4100\ kg$$

$$\text{tyre pressure} = 6\ kg/cm^2$$

and Allowable deflection, $\Delta = 5\ mm = 0.5\ cm$

By using Flexible plate formula.

$$P = A.p$$

$$A = \frac{P}{p} = \frac{4100}{6} = 683.33\ cm^2$$

∴ Radius of contact area

$$\therefore A = \pi r^2$$

$$\therefore \frac{A}{\pi} = r^2$$

$$\frac{683.33}{\pi} = r^2$$

$$\therefore r = 14.75\ cm$$

$$\Delta = 1.5\frac{P.a}{E_s}.F_2$$

$$0.5 = 1.5\times\frac{6\times 14.75}{35.4}.F_2$$

$$F_2 = 0.133\ mm$$

By using chart, $F_2 = 0.133\ ;\ \frac{E_s}{E_p} = \frac{1}{80}$

value of $\frac{h}{a} = 2.7$

∴ Thk. of pavement required

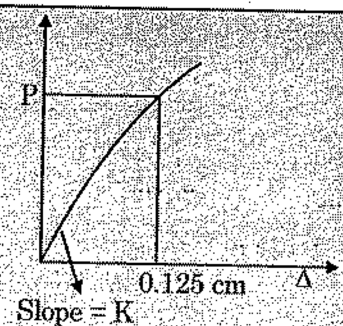
$$\therefore h = 2.7\times a = 2.7\times 14.75$$

$$h = 39.82\ cm$$

$$h = 40\ cm$$

DESIGN OF RIGID PAVEMENT

- The rigid pavement is constructed using cement concrete and the load carrying capacity in this case is primarily due to rigidity in the slab.
- Rigidity is due to high modulus of elasticity of concrete.
- Cement concrete pavement is rested on soil foundation which can be treated as a spring having spring constant K. This K is called modulus of subgrade reaction.
- Modulus of subgrade reaction is found out using plate loud test (plage bearing test).



$$\frac{P}{\Delta} = K$$

- Modulus of subgrade reaction is calculated corresponding to the settlement of 1.25 mm

$$\frac{P}{\Delta} = K = \frac{p\left(kg/cm^2\right)}{0.125\,cm}$$

$$K = \frac{p}{0.125}\ kg/cm^3$$

As per IRC, the modulus of sub-grade reaction corresponding to 75 cm plate used for testing is 1/2 of that using 30 cm plate.

$$\boxed{K_{75\,cm} = 0.5\,K_{30\,cm}}$$

Note : *K value also depends on the size of the plate used.*

$$\Delta = \frac{1.18pa}{E_S}$$

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

$$\frac{p}{K} = \frac{1.18pa}{E_S}$$

$$K.a. = \text{constant}$$

$$K_{75} \times 75 = K_{30} \times 30$$

$$K_{75} = \frac{30}{75} K_{30}$$

$$K_{75} = 0.4K_{30}$$

However IRC recommends $K_{75} = 0.5\ K_{30}$

- Normally for rigid pavement design test is done using 75 cm plate size.

Relative Stiffness of slab to subgrade

- Relative stiffness of slab with respect to subgrade is represented by radius of relative stiffness and it is given by

$$l\text{ (cm)} = \left[\frac{Eh^3}{12(1-\mu^2)K}\right]^{1/4}$$

E = Modulus of elasticity of cement concrete in kg/cm^2 (3×10^5 kg/cm^2).

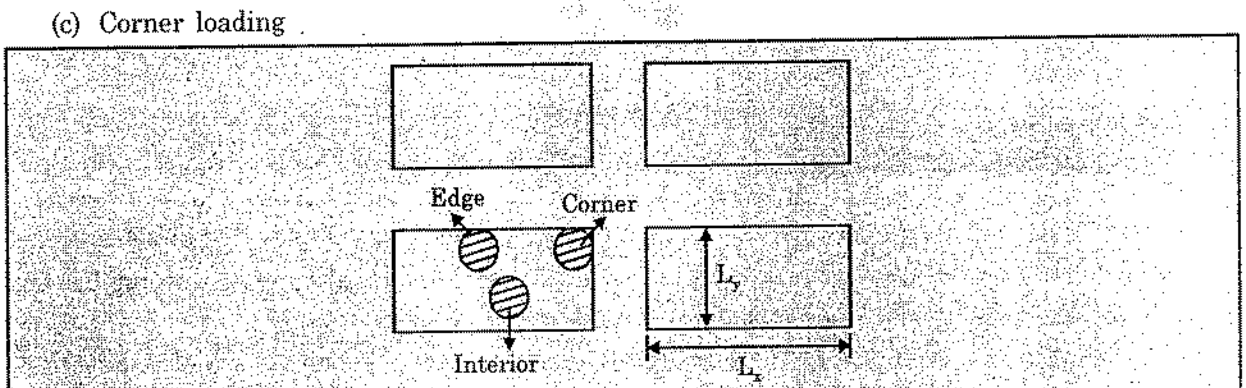
μ = poission ratio of concrete is taken as 0.15

h = slab thickness in cm

K = modulusof subgrade raction.

Critical Load Position

- Intensity of maximum stress in the slab depends on the location of load on the pavement.
- Three typical locations are considered critical. They are :
  (a) Interior loading
  (b) Edge loading
  (c) Corner loading



Equivalent Radius of Resisting Section

- Only a small area of the pavement is resisting a bending movement of a plate due to loading.
- Westergrad gave a relation for radius of resisting section in cm by

$$\boxed{b(\text{cm}) = b = \begin{cases} \sqrt{1.6a^2 + h^2} - 0.675h, & a < 1.724h \\ a & ,\text{otherwise} \end{cases}}$$

a = radius of wheel load distribution in cm.

h = slab thickness in cm

Westergard Stress Equation for Loading

- Westergard assumed winkler foundation (spring fondation) and the slab was assumed to be homogenous & isotropic.
- The stresses for internal edge and corner loadings are as given below.

$$\boxed{\underset{(\text{kg/cm}^2)}{\sigma_{\text{internal}}} = \frac{0.316P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right)+1.069\right]}$$

[σ_{internal} is the tensile stress at slab bottom]

$$\boxed{\underset{(\text{kg/cm}^2)}{\sigma_{\text{edge}}} = \frac{0.572P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right)+0.359\right]}$$

[σ_{edge} is the tensile stress at slab bottom]

$$\boxed{\underset{(\text{kg/cm}^2)}{\sigma_{\text{corner}}} = \frac{3P}{h^2}\left[1-\left(\frac{a\sqrt{2}}{l}\right)^{0.6}\right]}$$

[σ_{corner} is the tensile stress at top of slab]

P = Wheel load in kg

h = Thickness of slab in cm.

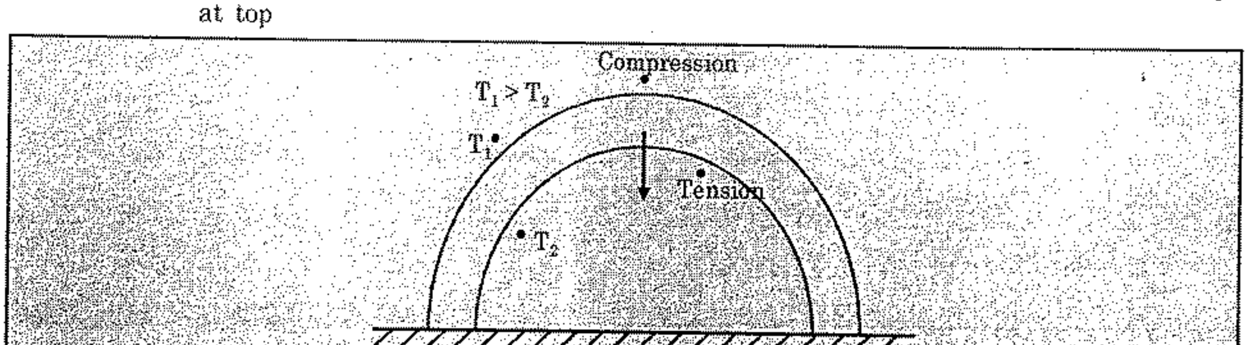
l = Radius of relative stiffness in cm.

b = Radius of resisting section in cm.

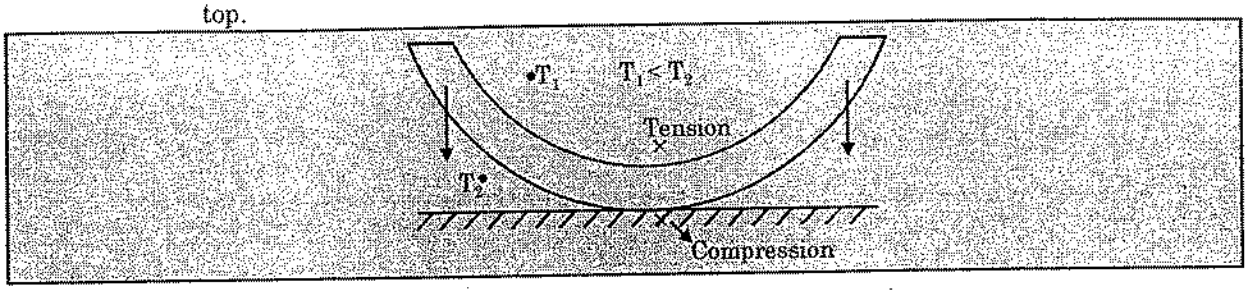
TEMPERATURE STRESSES

- Temperature stresses are developed in the cement concrete pavement due to variation in the slab temperature and resistance against deformation provided by the weight of slab and friction between slab & ground. The stresses are caused by
  (a) Daily variation resulting in temperature gradient across the thickness of slab.
  (b) Seasonal variation resulting in overall change in slab temperature.
- Daily variation will leads to warping of the slab. (Temperature deferential between the top & bottom of the slab causes curling i.e. warping stress in the pavement).

Day Time : Slab tries to expand at top but weight tries to resist it hence compression develops at top



Night Time : Slab tries to contract top but it is restrained by weight hence tension develops at top.



FORMULA FOR WARPING STRESSES

$$\sigma_{t\,internal} = \frac{E\alpha t}{2}\left(\frac{Cx + \mu Cy}{1-\mu^2}\right)$$

$$\sigma_{t\,edge} = \max\left\{\frac{CxE\alpha t}{2}, \frac{CyE\alpha t}{2}\right\}$$

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

E = modulus of elasticity of concrete = 3×10^5 kg/cm^2

α = thermal expansion co-efficient of concrete.

t = temperature differential between top and bottom of concrete slab.

Cx and Cy are co-efficient that depend upon $\frac{Lx}{l}, \frac{Ly}{l}$ respectively.

l = radius of relative stiffness.

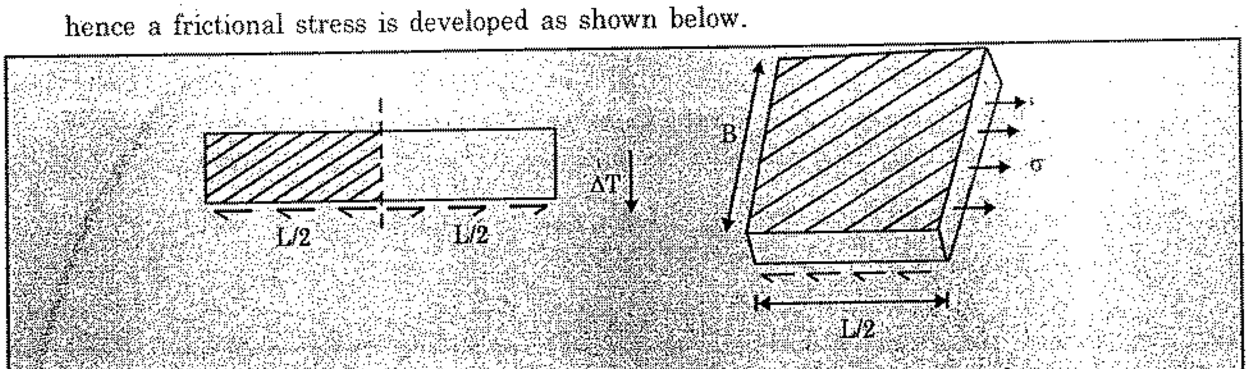
μ = possion's ratio is taken as 0.15

a = radius of contact

- These stresses are tensile stresses

FRICTIONAL STRESSLESS

- Frictional stresses are developed due to seasonal variation of temperature and in this case there is no temperature gradient across the thickness.
- If the slab tries to contract due to temperature fall, as in winter, the slab tries to move inward hence a frictional stress is developed as shown below.



$$\left[B \times \frac{L}{2} \times h\right]\gamma_{conc.}f = B \times h \times \sigma$$

$$\sigma = \frac{f.\gamma_{conc.}L}{2}$$

Note : *AS slab is trying to contract from both side hence half of the length of slab has been taken.*

- During summer compression develops in the clab

Critical Combination of Stresses

- Out of various wheel stresses.

1. Corner stress is maximum as there is discontinuity in both direction.
2. Interior stres is minimum
3. Edge stress is in intermediate range.

- Temperature stress is critical at the edge and interior and it is minimum at corner.

Note : *At the corner resistance due to weight is minimum, hence warping stress is minimum.*

- In combination of wheel load and temperature, edge region is most critical, hence designing is done using edge region stress and however checking is done for corner region.

Certain Combinations

1. Summer and Mid day

   $\sigma_{load\,edge} + \sigma_{warping\,edge} - \sigma_{friction}$ ....(α)

2. Winter Mid Day

   $\sigma_{load\,edge} + \sigma_{warping\,edge} + \sigma_{friction}$ ....(β)

3. Mid Night

   The critical combination of stress is for corner region and critical combination is given by

   $\sigma_{load\,corner} + \sigma_{warping\,corner}$ ....(γ)

Note : *Generally frictional stresses are assumed to be constant along length, but in reality it is not constant. It is zero at ends, and max at centre of slab. Hence in midnight combination we have not taken the effect of seasonal variation.*

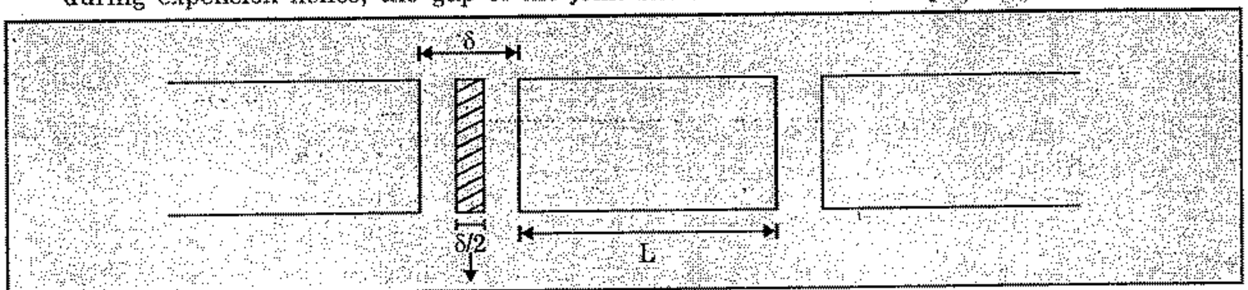
- 2nd combination becomes less than the 1st combination.

VARIOUS TYPES OF JOINTS PROVIDED IN THE CEMENT CONCRETE

- Various types of joints provided in the cement concrete pavement are :
  (a) Expansion joint
  (b) Contraction joint
  (c) Construction joint

(a) Expansion Joint

- The purpose is to allow the expansion of pavement due to rise in temperature with respect to construction temperature. To design the joint, we find out the joint spacing for a given joint thickness of 2.5 cm max as specified by IRC.
- The maximum spacing between expansion joint is 140 m.
- At the expansion joint, dowell are provided which develops bending, bearing and sharing stresses and help in load transfer.
- The fillers provided at the expansion joint are assumed to be compressed by 50% of its thickness during expension hence, the gap of the joint should be twice the expansion in concrete.



Filler has, the original thickness of "δ". But due to expansion of slab, it gets compressed to max of "$\delta/2$".

$$\Rightarrow \qquad L\,\alpha\,\Delta T = \frac{\delta}{2}$$

$$L = \frac{\delta}{2\alpha\Delta T}$$

L = Maximum spacing between expansion joint

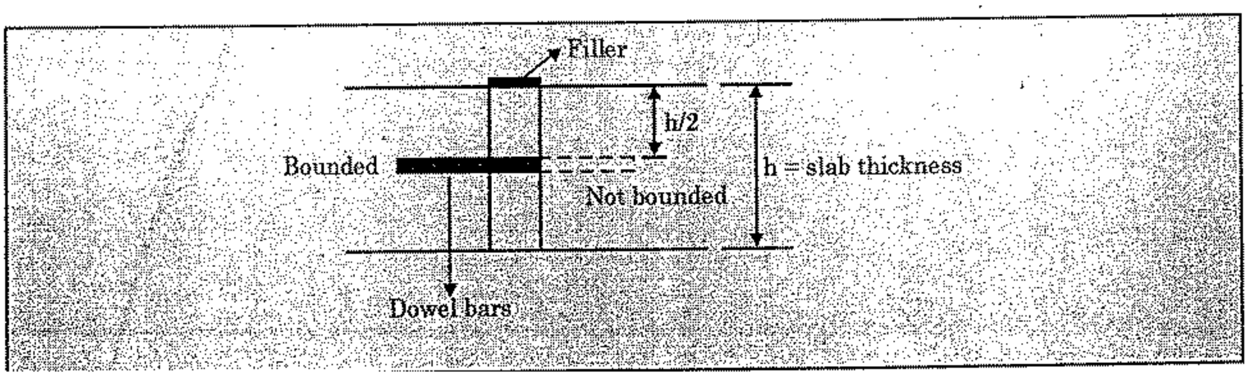
δ = gap of expansion joint

α = co-efficient of thermal expansion

ΔT = Rise in temperature.

(b) Contraction Joint

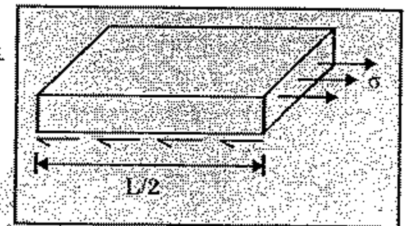
- It is provided to control crack due to shrinkage & moisture variation.
- To regulate the crack i.e. to ensure that crack forms at predetermined location, slab is weakend at certain intervals. These locations are called contraction joint.



Spacing of Contraction Joint

- During initial curing period shrinkage occurs in the concrete and if this shrinkage is resisted tensile stress develops in the concrete slab.
- Fall of temperature will also develop the tensile stress in the concrete slab.
- (a) When no reinforcement has been provided in the concrete slab; length of contraction joint is calculated as follows.

$$\sigma = \frac{fx\left[\gamma_{conc.} A \frac{L}{2}\right]}{A} = \frac{fL\gamma_{conc}}{2}$$



$$\Rightarrow \qquad \boxed{L = \frac{2\sigma}{f\,\gamma_{conc}}}$$

σ = permissible tensile stress in concrete.

f = Coefficient of friction between concrete & base

L = Spacing between contraction joint

γ_{conc} = unit wt. of concrete

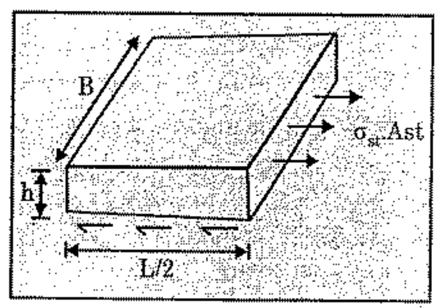
- When reinforcement is not provided, the maximum spacing between contraction joint is taken as 4.5m.

Note : $f = 1.5$; $\gamma_{conc} = 24\,KN/m^3$; If σ value is not given, it is taken as 0.8 kg/cm^2

(b) When reinforcement is provided in the slab it is assumed that all tension is taken by reinforcing steel.

$$\Rightarrow \qquad \sigma_{st}.A_{st} = B \times \frac{L}{2} \times h \times \gamma_{conc}.f$$

$$\boxed{L = \frac{2\sigma_{st} A_{st}}{Bh\gamma_{conc}.f}}$$

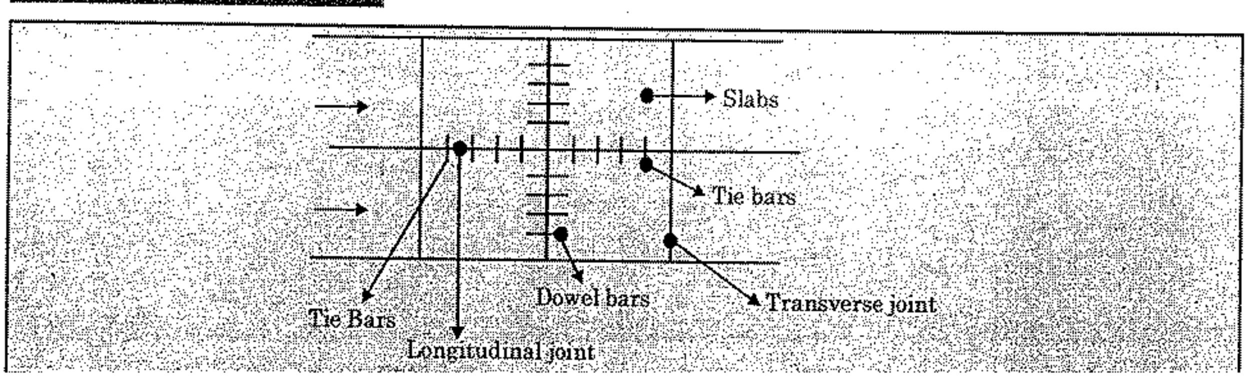


σ_{st} = permisisble tensile stress in steel.

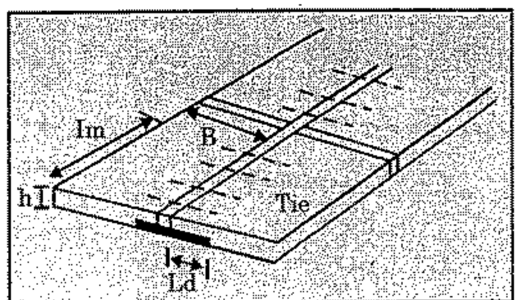
A_{st} = area of steel in complte width of slab.

f = Coefficient of friction.

LONGITUDINAL JOINT



- Longitudinal joints are provided along the length of pavement.
- It reduces the warping stress.
- The normal width of slab is generally 3 to 5m. If width of slab becomes more we invariably provide the two slabs and provide longitudinal joint.
- We generally provides tie bars at the longitudinal joint.
- The tie bars ensure that the two adjacent slab remains firmly together.
- Tie bars are not designed as load transfer device but ensures that the two slabs remain firmly together. Load is actually transferred to the adjacent slab due to aggregate interlock.



- Tie bars are bounded with concrete and we mostly used deformed bars of size approximately 10 mm. dia.
- Their length is smaller than the length of dowel bar. They are designed to withstand tensile stressed generated due to frictional forces between slab and the soil below.
- Assuming all forces in unit length of slab to be taken by the tie bars, we have

$$A_{st} \times \sigma_{st} = 1 \times B \times h\,\gamma_{conc.} f$$

$$A_{st} = \frac{Bhf.\gamma_{conc.}}{\sigma_{st}}$$

A_{st} = Area of Steel per unit length.

σ_{st} = permissible tensile stress in the steel (1400 kg/cm^2).

$f = 1.5$ = Coefficient of friction

γ_C = 24 kN/m^3

- The length of the tie bar is decided on the basis of development length.
- The length would be equal to 2 × development length.

$$\boxed{\text{length} = 2L_d}$$

Thus, $$n \times \pi \times \phi \times L_d \times \tau_{bd} = \frac{n \times \pi\phi^2}{4}\sigma st$$

$$L_d = \frac{\sigma_{st}\phi}{4\tau_{bd}}$$

$$\text{Length of tie} = \frac{\sigma_{st} \times \phi}{2\tau_{bd}}$$

Where n = no. of bars

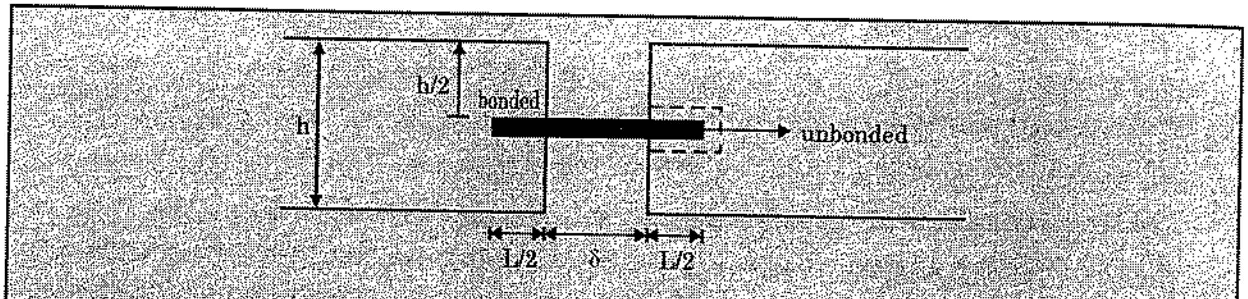
ϕ = dia. of the bar

L_d = development length

τ_{bd} = bond stress between steel bar & conc.

σ_{st} = permissible tensile stress in steel

DOWEL BAR (DESIGN)



Dowel Bars

- Dowel bars act as load transfer devices across transverse joint and they keep the two slab at the same height.
- They are mild steel round bars bonded on one side and free on the other side.
- Normally 25 mm to 40 mm dia and 400 and 500 mm length are provided.
- Stresses in the dowel bar are
  (a) Shear stress (b) Bending stress (c) Bearing stress
- The stresses in the dowel bar is given by Bradbury analysis.
- Load transfer capacity of single dowel bar is given by

1. **In shear**

$$P_S = \frac{\pi}{4} d^2 \times \sigma_S$$

Where

P_S = Load transfer capacity of dowel bar in shear.
d = Dia. of bowel bar
σ_S = Permissible Shearing Stress in Steel.

2. **In Bending**

$$P_b = \frac{2 \times d^3 \times \sigma_b}{L + 8.8\delta}$$

Where

d = Dia. of dowel base (cm)

σ_b = Permissible bending stress in dowel bar $\left(\text{kg/cm}^2\right)$

P_b = Load transfer capacity of dowel in bending (in kg)

δ = Gap of joint (cm)

L = Embedded length of dowel bar (cm).

3. **In Bearing**

$$P_{\text{bearing}} = \frac{\sigma_{br} L^2 d}{12.5(L + 1.5\delta)}$$

Where

$P_{bearing}$ = Load transfer capacities in bearing (kg).

L = Embedded length of dowel bar (cm)

d = Dia. of bar (cm)

δ = Gap of joint (cm)

σ_{br} = Permissible bearing stress.

Steps in the Design of Dowel Bar

Step 1 : Embedded length of the dowel bar is decided by equating strength in bending and bearing.

At this stage we assume the joint width (δ) and dowel diameter (ϕ).

$$L = 5d\sqrt{\frac{\sigma_b(L+1.5\delta)}{\sigma_{br}(L+8.8\delta)}}$$

Find L by assuming dia. of bar, (d) and joint width, (δ).

Step 2 : Find the load transfer capacity P_s, P_b, P_{br}

Step 3 :

- Load transfer capacity of the dowel bar system is assumed to be 40% of the wheel load.
- The distance on either side of the load position upto which the group of dowel bars are effective in load transfer is taken as $1.8\,l$.

  l = radius of relative stiffness.

Step 4 :

The load capacity factor required is calculate as,

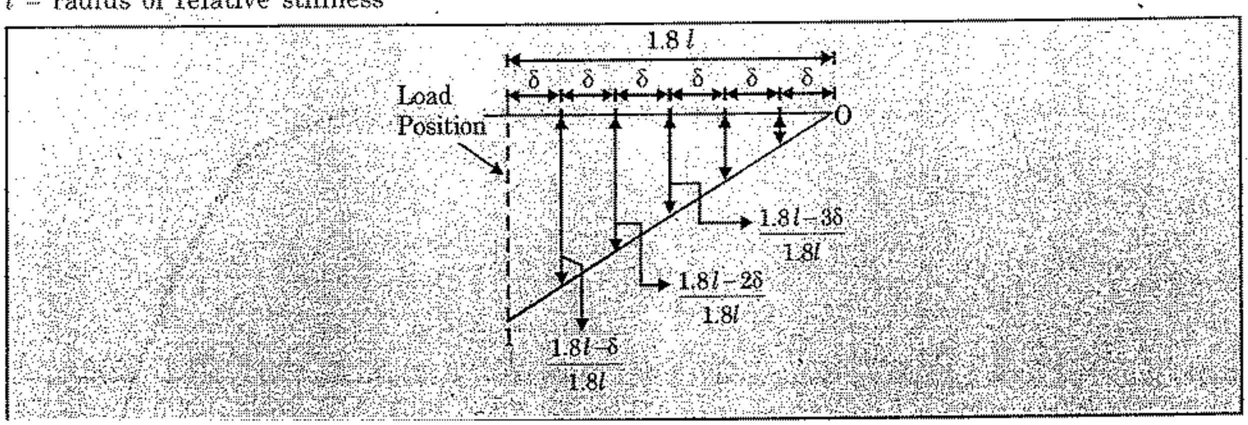
$$\max\left\{\frac{0.4P}{P_S}, \frac{0.4P}{P_b}, \frac{0.4P}{P_{br}}\right\}$$

Where, P = wheel load

Steps 5 :

The capacity factor of a dowel bar is assumed to be 1, just below the wheel and it is assumed to be zero at a distance of 1.8 l from the wheel.

l = radius of relative stiffness



$\Rightarrow$ $$\text{Load capacity} = 1 + \frac{(1.8l - \delta)}{1.8l} + \frac{(1.8l - 2\delta)}{1.8l} + \frac{(1.8l - 3\delta)}{1.8l} + \ldots\ldots$$

[So long as the numerator is +ve.]

- The spacing (δ) should be choosen such that the load capacity is greater than the load capacity factor.
- Actual length of dowel bar would be L + δ.

Example 17

Compute the equivalent radius of resisting section of 20 cm slab, given that the radius of contact area wheel load is 15 cm.

Sol. Given Data : h = 20 cm

Radius of contact area a = 15 cm

Since, $$\left(\frac{a}{h}\right) = \frac{15}{20} = 0.75 < 1.724$$

Therefore, $$b = \sqrt{\left(1.6a^2 + h^2\right)} - 0.675h = \sqrt{1.6 \times (15)^2 + (20)^2} - (0.675 \times 20) = 14.07\text{ cm}$$

Example 18

Compute the radius of relation stiffness of 15 cm thick cement concrete slab from the following data.

Modulus of elasticity of cement concrete = 210000 kg/cm^2

and Poisson's ratio for concrete = 0.13.

Modulus of subgrade reaction k = 3.0 kg/cm^3

k = 7.5 kg/cm^3

Sol. Given Data : h = 15 cm

Modulus of elasticity of cement concrete

$E = 21 \times 10^4$ kg/cm^2

$\mu = 0.13$

For k = 3.0 kg/cm^3

We know that

$$\text{Radius of relative stiffness } l = \left[\frac{Eh^3}{12k(1-\mu^2)}\right]^{1/4} = \left[\frac{21 \times 10^4 \times (15)^3}{12 \times 3 \times \left\{1 - (0.13)^2\right\}}\right]^{1/4} = 66.61\text{ cm.}$$

For k = 7.5 kg/cm^3

$$l = \left[\frac{Eh^3}{12k(1-\mu^2)}\right]^{1/4} = \left[\frac{21 \times 10^4 \times (15)^3}{12 \times 7.5 \times (1 - 0.13^2)}\right]^{1/4}$$

$$= 53.20\text{ cm}$$

Example 19

Calculate the stresses at interior edge and comer region of a cement concrete pavement using wastergaard's stream equation, using the following data :

Wheel load P = 4100 kg

Modulus of Elasticity of cement concrete $E = 3.3 \times 10^5$ Kg/cm²

Poisson's ratio of subgrade $\mu = 0.15$

Modulus of subgrade reaction $K = 25$ kg/cm³

Radius of contact area a = 12 cm

Pavement thickness h = 18 cm

Sol. Given Data : Wheel load P = 4100 kg

Modulus of Elasticity of cement concrete $E = 3.3 \times 10^5$ Kg/cm²

Poission's ratio of subgrade $\mu = 0.15$

$K = 25$ kg/cm³

a = 12 cm

h = 18 cm

We know that

$$\text{Radius of relative stiffness } l = \left[\frac{Eh^3}{12k(1-\mu^2)}\right]^{1/4}$$

$$= \left[\frac{3.3\times10^5\times(18)^3}{12\times25\times(1-0.15^2)}\right]^{1/4} = 50.61 \text{ cm}$$

Radius of equivalent resisting section

$$b = \sqrt{1.6a^2+h^2} - 0.675h$$

$$= \sqrt{1.6\times(12)^2+(18)^2} - (0.675\times18) = 11.39 \text{ cm}$$

Stress at the interior

$$\sigma_i = \frac{0.316}{h^2}\times P\left[4\log_{10}\left(\frac{l}{b}\right)+1.069\right] = \frac{0.316\times4100}{(18)^2}\left[4\log_{10}\left(\frac{50.61}{11.39}\right)+1.069\right] = 14.63 \text{ kg/cm}^2$$

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

$$= \frac{0.572\times4100}{(18)^2}\left[4\times\log_{10}\left(\frac{50.61}{11.39}\right)+0.359\right] = 21.35 \text{ kg/cm}^2$$

Stress at the corner (σ_c)

$$= \frac{3P}{h^2}\left[1-\left(\frac{a\sqrt{2}}{l}\right)^{0.6}\right] = \frac{3\times4100}{(18)^2}\left[1-\left(\frac{12\times\sqrt{2}}{50.61}\right)^{0.6}\right] = 18.25 \text{ Kg/cm}^2$$

Example 20

Determine the warping stresses at interior Edge and corner region in a 32 cm. thick concrete pavement with transverse joints and longitudinal joints at 4.5 m and 3.6 m interval respectively. The modulus of subgrade reaction is 6 kg/cm³. Assume temperature differential for a day condition to be 15°C and radius of contact area in 15 cm additional data for cement concrete pavement is given below:

$l = 10\times10^{-6}$ per°C $E = 3\times10^5$ kg/cm² $\mu = 0.15$ $C_x = 0.80$ $C_y = 0.45$

Sol. Given Data : $t = 15°C$

Radius of contact area $a = 15$ cm

$$\varepsilon = 10\times10^{-6} \text{ per°C}$$

$$E = 3\times10^5 \text{ kg/cm}^2 \text{ ; } \mu = 0.15$$

$$C_x = 0.80 \text{ and } C_y = 0.45$$

$$k = 6 \text{ kg/cm}^3$$

$$h = 32 \text{ cm}$$

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

$$= \frac{3\times10^5\times10\times10^{-6}\times15}{2}\left[\frac{0.80+0.15\times0.45}{1-(0.15)^2}\right] = 19.96 \text{ kg/cm}^2$$

Radius of relative stiffness

$$l = \left[\frac{Eh^3}{12k(1-\mu^2)}\right]^{1/4} = \left[\frac{3\times10^5\times(32)^3}{12\times6\times(1-0.15^2)}\right]^{1/4} = 108.71 \text{ cm}$$

Warping stream at corner region

$$\sigma_{t(c)} = \frac{E\alpha t}{3(1-\mu)}\sqrt{\left(\frac{a}{l}\right)} = \frac{3\times10^5\times10\times10^{-6}\times15}{3\times(1-0.15)}\sqrt{\frac{15}{108.71}} = 6.56 \text{ kg/cm}^2$$

Example 21

Design a dowel bar systm for the following condition.

Design wheel load = 51 kN

Design load transfer = 40%

Slab thickness = 28 cm

Joint width (S) = 2 cm

Permissible flexural stress in dowel bar = 140 MN/m²

Permissible shear stress in bar = 100 MN/m²

Permissible bearing stress on concrete = 10 MN/m²

k value of subgrade = 80 MN/m³

$$E = 30{,}000 \text{ MN/m}^2$$

$$M = 0.15$$

Sol. Calculation of l

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

$$= \left[\frac{30{,}000\times(0.28)^3}{12\times80(1-0.15^2)}\right]^{\frac{1}{4}}$$

$$l = 0.915 \text{ m} \qquad \text{(i)}$$

Calculation of dowel length

Assume d = 2.5 cm. = 0.025 m

$$L_d = 5d\left[\frac{F_f}{F_b}\times\frac{L_d+1.5\delta}{L_d+8.8\delta}\right]^{\frac{1}{2}}$$

$$L_d = 5\times0.025\left[\frac{140}{10}\times\frac{L_d+1.5\times0.02}{L_d+8.8\times0.02}\right]^{\frac{1}{2}}$$

⇒ By trial and error

$$L_d = 0.405 \qquad \text{(ii)}$$

Dowel length = (0.405 + 0.02) = 0.425 m

Say = 0.45 m

Load transfer capacity of single dowel

$$\bar{P}(\text{Shear}) = 0.785\times(0.025)^2\times100 \text{ MN}$$

$$= 0.0490 \text{ MN}$$

$$= 49 \text{ kN} \qquad \text{(A)}$$

$$\bar{P}(\text{bending}) = \frac{2d^3F_f}{(L_d+8.8\delta)} = \frac{2\times(0.025)^3\times140}{(0.4050+8.8\times.02)}$$

$$= 7.53\times10^{-3} \text{ MN} = 7.53 \text{ kN} \qquad \text{(B)}$$

$$\bar{P}(\text{bearing}) = \frac{(F_b\times L_d^2\times d)}{12.5(L_d+1.5\delta)}$$

$$= \frac{10\times(.405)^2\times.025}{12.5(0.405+1.5\times.02)} = 7.54\times10^{-3} \text{ MN}$$

$$= 7.54 \text{ kN} \qquad \text{(C)}$$

Minimum of (A), (B), and (C) should be governing

$$P' = 7.53 \text{ kN}$$

Capacity factor required for load transfer :

∴ Load transfer = 0.40 × 51 = 20.4 kN

∴ Required capacity factor = $\frac{20.4}{(7.53)}$ = 2.71

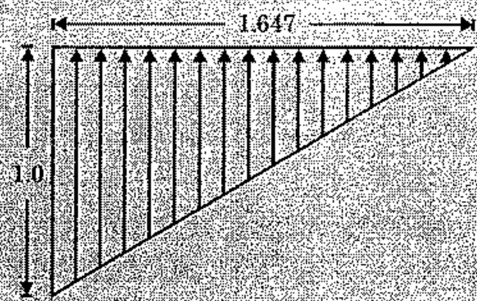
Spacing of dowel bar

∴ Distance over which dowel is effective =

$$1.8\,l = (1.8\times0.915)$$

$$= 1.647 \text{ m}$$

Assuming a dowel spacing of 0.3 m



$$\text{Available capacity factor} = 1+\left(\frac{1.647-0.3}{1.647}\right)+\left(\frac{1.647-0.6}{1.647}\right)+\left(\frac{1.647-0.9}{1.647}\right)+\left(\frac{1.647-1.2}{1.647}\right)$$
$$= 3.18$$

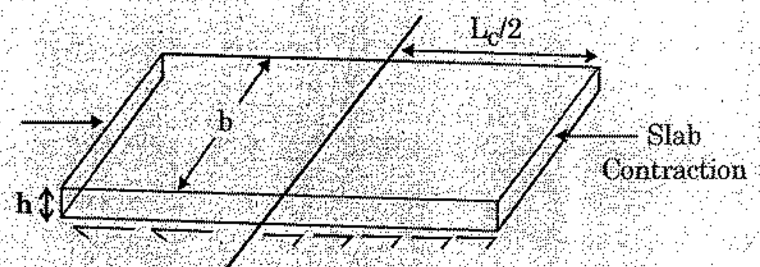
Which is higher then the required capacity factor of 2.71

Hence (o.k.)

Example 22

What total cross sectional area of longitudinal reinforcing steel is required for a cement concrete pavement slab 3.75 m wide, 15 m long and 200 mm thick? The concrete weighs 2400 kg/m³. Working stress in steel is 1400 kg/cm². Coefficient of friction = 1.5.

Sol.



Assume that the reinforcement takes the entire tensile force in the slab, caused by the frictional resistance of subgrade.

Therefore, Tensile Force = Frictional Resistance

$$\Rightarrow \quad A_{st} \times \sigma_{st} = f \times N = f \times b \times \left(\frac{L_C}{2}\right) \times \left(\frac{h}{100}\right) \times W$$

$$A_{st} = \frac{(b \times h \times L_C)}{(200 \times \sigma_{st})} \quad \dots \quad \text{(i)}$$

where, b = Distance between joint and nearest free edge Inm.

h = thickness of pavement In cm,

W = unit weight of concrete in Kg/m³

σ_{st} = allowable working stress in tension for steel in kg/cm²

L_C = length of slab in meter,

f = coefficient of friction between pavement and subgrade.

$$A_{st} = \frac{(3.75 \times 20 \times 1.5 \times 2400)}{(200 \times 1400)} = 14.46 \text{ cm}^2$$

Example 23

A cement concrete pavement has a thickness of 24 cm and has two lanes of total width 7.2 m, with longitudinal joint. Design the dimension and spacing of the bars, using the folloiwng data

(i) Allowable working stress in steel in tension = 1400 kg/cm²

(ii) Unit weight of concrete = 2400 kgm³

(iii) Coefficient of friction = 1.5

(iv) Allowable bond stress in concrete = 24.6 kg/cm²

Sol. Assume that the reinforcement takes the entire tensile.

Force in the slab, caused by the frictional resistance of subgrade,

Therefore, per meter length of slab the longitudinal reinforcement is given by

$$A_{st} = \frac{(b \times h \times f \times W)}{100 \times \sigma_{st}}$$

where, A_{st} = Area of longitudinal steel in cm², f = frictional coefficient

b = distance between joint and nearest free edge to m, h = thickness of slab in cm

σ_{st} = Allowable working stress in steel in tension in kg/cm²

W = Unit weight of concrete in kg/m³

Given Data : f = 1.5 ; $b = \frac{7.2}{2} = 3.6$ m ; h = 24 cm ; W = 2400 kg/m³

$$\sigma_{st} = 1400 \text{ kg/cm}^2$$

$$A_{st} = \frac{1.5 \times 3.6 \times 24 \times 2400}{(100 \times 1400)} = 2.22 \text{ cm}^2$$

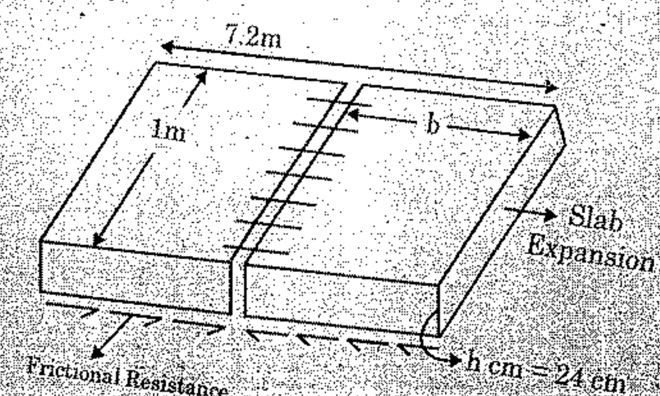
Assuming diameter of tie bar = 1 cm

$$\text{Number of tie bars} = \frac{2.22}{\frac{\pi}{4} \times (1)^2} = 2.83$$

$$\text{Spacing of tie bar} = \left(\frac{1000}{2.83}\right) = 353.78 \text{ mm say } 353 \text{ mm}$$

Now, length of tie bar $L_t = \frac{d \times \sigma_{st}}{2 \times (S_b)} = \frac{1 \times 1400}{2 \times 24.6} = 28.5$ cm say 30 cm

∴ 1cm diameter bars of length 30 cm at a spacing of 353 c/c can be provided.



Example 24

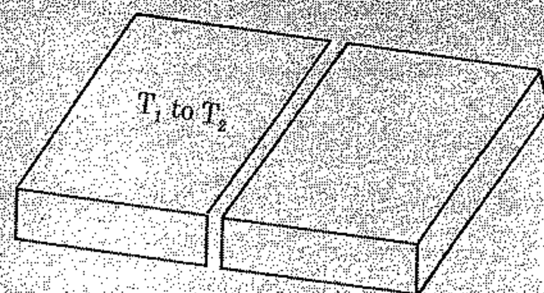
The width of expansion joint gap is 2.5 cm. In a cement concrete pavement. If the lagging temperature is 10°C and maximum slab temperature in summer is 54°C, calculate the spacing between expansion joints. Assume coefficient of thermal expansion of conrete as 10×10^{-6} per°C.

Sol. Data Given : $T_1 = 10°C$

$T_2 = 54°C$

The joint Filler may be assumed to be compressed upto 50% of its thickness

Therefore, $\delta' = \frac{\delta}{2} = \left(\frac{2.5}{2}\right) = 1.25\,\text{cm}$



$$\delta' = L_e \alpha (T_2 - T_1)$$

$$\Rightarrow \quad 1.25\times10^{-2} = L_e \times 10\times10^{-6}(54-10)$$

$$\therefore \quad L_e = \frac{1.25\times10^{-2}}{(10\times10^{-6}\times44)} = 28.409\,\text{m}$$

Example 25

Determine the spacing between contraction joints for 3.5 m slab width having thickness of 20 cm and f = 1.5. For the following two cases :

(i) For plain cement concrete, allowable $S_c = 0.8$ kg/cm²

(ii) For Reinforcement cement concrete, 1.0 cm diameter, bars at 0.30 m spacing

Sol. (i) For Plain Cement Concrete Slab (Without Reinforcement)

We know that

$$\Rightarrow \quad \left(\cancel{b}\times\left(\frac{L_C}{2}\right)\times\frac{\cancel{h}}{100}\times\gamma\right)\times f = S_c\times\cancel{b}\times\cancel{h}\times100$$

$$\therefore \quad L_C = \left(\frac{2S_c\times10^4}{\gamma\times f}\right) = \frac{2\times0.8\times10^4}{(2400\times1.5)} = 4.44\,\text{m}$$

(ii) Total cross sectional area of steel (A_s) in one direction along the slab width

$$A_{st} = \left(\text{number of bars}\times\frac{\pi}{4}d^2\right)$$

$$= \left(\frac{3.5}{0.3}\right)\times\frac{\pi}{4}\times(1)^2 = 9.16\,\text{cm}^2$$

Spacing between contraction joints

$$L_C = \frac{200\times A_{st}\times6_{st}}{(b\times h\gamma\times f)}$$

$$= \frac{200\times1200\times9.16}{(3.5\times20\times2400\times1.5)} = 8.72\,\text{m}$$

OBJECTIVE QUESTIONS

1. Given that:

   r = radius of load distribution

   E = modulus of elasticity of concrete

   K = modulus of subgrade reaction

   μ = Poisson's ratio of concrete

   h = thickness of slab

   P = wheel load

   The combination of parameters required for obtaining the radius of relative stiffness of cement concrete slab is

   (a) E, K, μ, r (b) h, K, μ, r

   (c) E, h, K, μ (d) P, h, K, μ

2. Effect of impact on the design of rigid pavements is accounted for by

   (a) increasing the thickness as would calculated with static wheel load

   (b) providing a base course

   (c) adopting a reduced flexural strength of concrete through a factor of safety

   (d) adopting an increased stress relative to that produced by static wheel load

3. Consider the following statements with reference to pavements:

   1. Flexible pavements are more suitable than rigid pavements in regions where subgrade strength is uneven.
   2. Load carrying capacity of rigid pavements depends more on the properties of concrete than the strength of subgrade.
   3. Compared to flexible pavements, rigid pavements are more affected by temperature variations.

   Which of these statements is/are correct?

   (a) 1 and 2 (b) 1 and 3

   (c) 2 and 3 (d) 3 alone

4. The number of load cycles (N_f) to cause the failure of a pavement is proportional to (P is the respective applied load)

   (a) P^4 (b) P^{-4}

   (c) P^2 (d) P^{-1}

5. **Assertion (A):** Load stresses added to warping stresses are considered while determining the thickness of pavement.

   Reason (R): Joints and sealing devices are used to relieve and/or take care of warping stresses, and the design for thickness is accordingly base upon load alone.

6. The general requirements in constructing a reinforced concrete road is to place a single layer of reinforcement

   (a) near the bottom of the slab (b) near the top of the slab

   (c) at the middle (d) equally distributed at the top and bottom

7. As per latest IRC guidelines for designing flexible pavement of CBR method, the load parameter required is
   (a) number of commercial vehicles per day
   (b) cumulative standard axles in msa
   (c) equivalent single axle load
   (d) number of vehicles (all types) during design life

8. If the load, warping and frictional stresses in a cement concrete slab are 210 N/mm^2, 290 N/mm^2 and 10 N/mm^2 respectively, the critical combination of stresses during summer mid-day is
   (a) 290 N/mm^2 (b) 390 N/mm^2
   (c) 490 N/mm^2 (d) 590 N/mm^2

9. In cement concrete pavements, tie bars are installed in
   (a) expansion joints (b) contraction joints
   (c) warping joints (d) longitudinal joints

10. **Assertion (A):** The critical combination of stresses on a cement concrete pavement during summer is given by load stress – warping stress + frictional stress.
    Reason (R): The critical combination of stresses in a cement concrete pavement during winter is given by: load stress + warping stress + frictional stress.

11. Consider the following factors:
    1. Period of construction, winter/summer
    2. Degree of foundation roughness
    3. Slab thickness
    4. Reinforced/unreinforced

    Which of these factors are considered as per IRC for obtaining the maximum expansion joint spacing in rigid pavements?
    (a) 1, 2 and 3 (b) 2, 3 and 4
    (c) 2 and 3 (d) 1 and 4

12. Which of the following factors have to be considered for the design of the flexible pavement for a highway?
    1. Design wheel load
    2. Strength of pavement component material
    3. Expansion joints

    Select the correct answer using the codes given below:
    (a) 1, 2 and 3 (b) 2, 3 and 4
    (c) 1, 3 and 4 (d) 1, 2 and 4

13. Match **List-I** (Method of design for flexible pavement) with **List-II** (Principle) and select the correct answer using the codes given below the lists:

    List-I
    A. Group Index Method
    B. CBR Method
    C. US Navy Method

D. Asphalt Institute Method

List-II

1. Semi-theoretical

2. Quasi-rational

3. Empirical method using soil classification test

4. Empirical method using soil strength test

Codes:

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

14. Which of the following factors are used for calculating temperature stress at the critical edge region in rigid pavement design?

1. Maximum temperature difference between summer and winter
2. Coefficient of thermal expansion of concrete
3. Slab length
4. Slab width

Select the correct answer using the codes given below:

(a) 1, 2 and 3 (b) 2, 3 and 4

(c) 1 and 2 only (d) 1 and 3

15. IRC code No. 37-1985 deals with which one of the following?

(a) Design of rigid pavements, taking ESWL and CBR into account.

(b) Design of rigid pavements, taking axle load and CBR into account.

(c) Design of flexible pavement, taking ESWL and CBR into account.

(d) Design of flexible pavement taking cumulative axle loads and CBR into account.

16. Radius of relative stiffness of cement concrete pavement does not depend upon which one of the following?

(a) Modulus of subgrade reaction

(b) Wheel load

(c) Modulus of elasticity of cement concrete

(d) Poisson's ratio of concrete

17. For conditions obtaining in India, at which location in a cement concrete pavement will the combined stresses due to traffic wheel load and temperature have to be critically checked during design?

(a) Corner (b) Corner and interior

(c) Corner and edge (d) Corner, edge and interior

18. Which one of the following sets of factors is related to design of thickness rigid pavement by Westergaard method?

(a) CBR value and stiffness index of soil

(b) Deflection factor and traffic index
(c) Swelling index and bulk modulus
(d) Radius of relative stiffness and modulus of subgrade reaction

19. Consider the following in relationto group index of soil:
1. Liquid limit
2. Sandy loam
3. Plasticity index
4. Percent passing 75 microns sieve

Which of the above is/are used for estimating the group index?
(a) 1 only (b) 1 and 2
(c) 2 and 3 (d) 1, 3 and 4

20. Assertion (A): Dowel bars are provided at expansion joints and sometimes also at contrction joints in cement concrete slabs.
Reason (R): Longitudinal joints in cement concrete pavements are constructed with tie bars.

21. Assertion (A): Tie bars are used in cement concrete slabs across the longitudinal joints.
Reason (R): Tie bars are designed to act as load transfer devices.

22. In cement concrete pavements, tie bars are provided
(a) near the top of slab across expansion joints
(b) near the bottom of slab across contraction joints
(c) at mid depth of slab across longitudinal joints
(d) near the bottom of slab across longitudinal joints

23. Consider the following parameters:
1. Radius of equivalent distribution of pressure
2. Width of slab
3. Spacing between contraction joints
4. Spacing between expansion joints
5. Radius of relative stiffness

Which of these parameters are taken into account for computing Bradbury's coefficient for temperature stresses in Westergaard's analysis for rigid pavement design?
(a) 1, 3, 4 and 5 (b) 2, 3 and 4
(c) 2, 3 and 5 (d) 1, 2 and 4

24. Consider the following factors:
1. Magnitude of load
2. Thickness of cement concrete slab
3. Temperature distribution in the slab
4. Modulus of subgrade reaction

Which of these should be taken into reckoning to determine the wheel load stress at critical location in a cement concrete pavement?
(a) 1, 2 and 3 (b) 1 and 3
(c) 3 and 4 (d) 1, 2 and 4

25. If the pressure carried by a CBR specimen at 2.5 mm penetration is 3.5 N/mm^2, the CBR of the soil is

(a) 10%
(b) 35%
(c) 50%
(d) 70%

26. What shall be the density in kg/m^3 of bituminous concrete with 6% bitumen and 3% air voids (Specific gravity of aggregate and bitumen are 2.65 and 1.01 respectively)?

(a) 2603
(b) 2415
(c) 2342
(d) 2303

27. If the modulus of sub-grade reaction of standard plate of 30 cm diameter is 16 kg/cm^3, the value for a standard plate of 75 cm diameter is

(a) 6.4 kg/m^3
(b) 7.4 kg/cm^3
(c) 8.0 kg/cm^3
(d) 9.3 kg/cm^3

28. Given that:

r = radius of load distribution
E = modulus of elasticity of concrete
K = modulus of subgrade reaction
μ = Poisson's ratio of concrete
h = thickness of slab
P = wheel load

The combination of parameters required for obtaining the radius of relative stiffness of a cement concrete slab is

(a) E, K, μ, r
(b) E, h, K, μ
(c) h, K, μ, r
(d) Ph, h, K, μ

29. Which one of the following statements is correct with respect to rigid pavements?

(a) Vertical stresses are transmitted to the lower layer through the points of contact in granular structure
(b) Vertical compressive stress is maximum on the pavement surface directly under the wheel load
(c) Lower layers have to take lesser magnitudes of stress
(d) Stresses on rigid pavements are transmitted to a wider area of sub-grade.

30. Consider the following statements:

Difference between airport and highway pavement thickness arises due to

1. magnitude of wheel load
2. tyre pressure
3. number of repetitions
4. size of the vehicle

Which of these statements are correct?

(a) 1, 2 and 4
(b) 1, 2, and 3
(c) 1 and 4
(d) 2, 3 and 4

31. Consider the following statements:
In the design of modern concrete pavements, the bars are used
1. as load transfer devices
2. in expansion joints
3. in contraction joints
4. in warping joints

Which of these statements is/are correct?
(a) 1 and 2
(b) 2 and 4
(c) 1 only
(d) 2 only

32. Which of the following stress combinations are appropriate in identifying the critical condition for the design of concrete pavements?

Type of stress	**Location**
A. Load	**1.** Corner
B. Temperature	**2.** Edge
	3. Interior

Select the correct answer using the codes given below:
(a) A-2, B-3
(b) A-1, B-3
(c) A-3, B-1
(d) A-2, B-2

33. It is proposed to widen and strengthen an existing 2-lane NH section as a divided highway. The existing traffic in one direction is 2500 commercial vehicles (CV) per day. The construction will take 1 year. The design CBR of soil subgrade is found to be 5 per cent. Given: traffic growth rate for CV = 8 per cent, vehicle damage factor = 3.5 (standard axles per CV), design life = 10 years and traffic distribution factor = 0.75. The cumulative standard axles (msa) computed are
(a) 35
(b) 37
(c) 65
(d) 70

34. Which of the following are the purpose for use of steel bar reinforcement in cement concrete pavements?
1. To increase the flexural strength of concrete
2. To prevent the onset of cracks
3. To allow wider spacing of joints

Select the correct answer using the codes given below:
(a) 1 and 2
(b) 2 and 3
(c) 1 and 3
(d) 1, 2 and 3

35. Match **List-I** (Item) with **List-II** (Use) and select the correct answer using the codes given below the lists:

List-I
A. O and D Survey
B. Collision diagram
C. OMC
D. Radius of relative stiffness

List-II
1. Concrete pavement design
2. Compaction

3. Accident studies
4. Traffic Survey

Codes:

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

36. What should be the rigidity factor for a tyre pressure greater than 7.0 kg/cm^2?

(a) Equal to 1 (b) Less than 1
(c) Between 1.1 and 2.0 (d) More than 2

37. Given a wheel load, what is the stipulated number of repetitions of load for which the enabling pavement is to be designed?

(a) 1000 (b) 10000
(c) 100000 (d) 1000000

38. In the context of flexible pavement design, the ratio of contact pressure to tyre pressure is called the Rigidity Factor. This factor is less than unity when the tyre pressure is

(a) less than 0.56 N/mm^2 (b) equal to 0.56 N/mm^2
(c) equal to 0.7 N/mm^2 (d) more than 0.7 N/mm^2

39. The data given below pertain to the design of a flexible pavement:

Initial traffic = 1 2 1 3 cvpd
Traffic growth rate = 8% per annum
Design life = 12 years
Vehicle damage factor = 2.5
Distribution factor = 1.0
The design traffic in terms of million standard axles (msa) to be catered would be

(a) 0.06 msa (b) 8.40 msa
(c) 21.00 msa (d) 32.26 msa

40. For a 25 cm thick cement concrete pavement, analysis of stresses gives the following values:

Wheel load stress due to corner loading.....................30 kg/cm^2
Wheel load stress due to edge loading.....................32 kg/cm^2
Warping stress at corner region during summer.....................9 kg/cm^2
Warping stress at corner region during winter.....................7 kg/cm^2
Warping stress at edge region during summer.....................8 kg/cm^2
Warping stress at edge region during winter.....................6 kg/cm^2
Frictional stress during summer.....................5 kg/cm^2
Frictional stress during winter.....................4 kg/cm^2
The most critical stress value for this pavement is

(a) 40 kg/cm^2 (b) 42 kg/cm^2
(c) 44 kg/cm^2 (d) 45 kg/cm^2

41. The following observations were made of an axle-load survey on a road:

Axle load (kN)	Repetitions per day
35-45	800
75-85	400

The standard axle-load is 80 kN. Equivalent daily number of repetitions for the standard axle-load are

(a) 450 (b) 480
(c) 800 (d) 1200

42. In case of governing equations for calculating wheel load stresses using Westergaard's approach, the following statements are made:

1. Load stresses are inversely proportional to wheel load.
2. Modulus of subgrade reaction is useful for load stress calculation.

(a) Both statements are TRUE (b) 1 is TRUE and 2 is FLASE
(c) Both statements are FALSE (d) 1 is FALSE and 2 is TRUE

43. Using **IRC:37-1984** "Guidelines for the Design of Flexible Pavements" and the following data, choose the total thickness of the pavement.

Number of commercial vehicles when construction is completed	2723 veh/day
Annual growth rate of the traffic	5.0%
Design life of the pavement	10 years
Vehicle damage factor	2.4
CBR value of the subgrade soil	5%

Data for 5% CBR value	
Number of Standard Axles, msa	Total thickness, mm
20	620
25	640
30	670
40	700

(a) 620 mm (b) 640 mm
(c) 670 mm (d) 700 mm

44. The width of the expansion joint is 20 mm in a cement concrete pavement. The laying temperature is 20°C and the maximum slab temperature in summer is 60°C. The coefficient of thermal expansion of concrete is 10×10^{-6} mm/mm/°C and the joint filler compresses up to 50% of the thickness. The spacing between expansion joints should be

(a) 20 m (b) 25 m
(c) 30 m (d) 40 m

45. The following data pertains to the number of commercial vehicles per day for the design of a flexible pavement for a national highway as per **IRC:37-1984**:

Type of commercial vehicle	Number of vehicle per day considering the number of lanes	Vehicle damage factor
Two axle trucks	2000	5
Tandem axle trucks	200	6

Assuming a traffic growth factor of 7.5% per annum for both the types of vehicles, the cumulative number of standard axle load repetitions (in million) for a design life of ten years is

(a) 44.6 (b) 57.8
(c) 62.4 (d) 78.7

46. What are the maximum value of CBR and minimum value of GI of any material, respectively?

(a) 100, 0 (b) 100, 20
(c) 50, 5 (d) 10, 0

47. Which one of the following methods is used in the design of rigid pavements?

(a) CBR method (b) Group index method
(c) Westergaard's method (d) McLeod's method

48. The subgrade soil properties of a sample are as follows:

Soil portion passing 0.075 mm sieve = 50%

Liquid limit = 40%

Plasticity index = 20%

The group index of the soil is

(a) zero (b) 4
(c) 6.5 (d) 8

49. What does the CBR pavement design curve give?

(a) Equivalent single wheel load
(b) Thickness of the overlay
(c) Thickness of the pavement cover to be provided
(d) Thickness of granular sub-base course

ANSWERS

1.	(c)	2.	(a)	3.	(c)	4.	(d)	5.	(b)	6.	(c)	7.	(b)
8.	(c)	9.	(d)	10.	(d)	11.	(a)	12.	(d)	13.	(c)	14.	(b)
15.	(d)	16.	(h)	17.	(c)	18.	(d)	19.	(d)	20.	(b)	21.	(c)
22.	(c)	23.	(c)	24.	(d)	25.	(c)	26.	(c)	27.	(a)	28.	(b)
29.	(b)	30.	(b)	31.	(b)	32.	(d)	33.	(d)	34.	(c)	35.	(b)
36.	(b)	37.	(d)	38.	(d)	39.	(c)	40.	(b)	41.	(b)	42.	(d)
43.	(c)	44.	(b)	45.	(b)	46.	(a)	47.	(c)	48.	(c)	49.	(c)

HINTS AND SOLUTIONS

1. (c)

Sol. The Radius of Relative Stiffness is given by

$$l = \left[\frac{Eh^3}{12k(1-\mu^2)}\right]^{\frac{1}{4}} - I$$

For obtaining the radium of Relative Stiffness E, h, k, u parameters are required.

2. (a)

Sol. Effect of impact on the design of Rigid pavements is accomted for by imerasing the thickness as would calculated with static wheel load.

3. (c)

Sol. Load carrying capacity of Rigid pavements depends more on the properties of concete hram the strength of subgrade.

Compared to flexible pavements, rigid pavements are more affected by temperature variation.

Hence 2 and 3 statements are correct.

4. (d)

Sol. $N_f P = \text{Constant}$

$$\boxed{N_f \, \alpha \, P^{-1}}$$

5. (b)

Sol.

6. (c)

Sol. The general equipments in constructing a reinforced concrete road is to place a single layer of Reinforcement at the middle.

8.

Sol. During summer mid day

The critical combination of stresses

$= 210 + 290 - 10 = 490 \text{ N/mm}^2$

9. (d)

Sol. • In cement concrete pavements, Tie Bars are installed in longitudinal joints

• Tie Bars are provided for load transfer.

10. • Critical combination of stress on a cement concrete pavement during summer is given by

= Load Stress + Warping Stress – Frictional Stress

• Critical combination of Stress on a cement concrete pavement during winter is given by

= Load Stress + Warping Stress + Frictional Stress

11. (a)

12. (d)

Sol. Design of flexible pavement, factors to be considered for a highway

(i) Design wheel load

(ii) Strength of pavement component material

(iii) Rainfall intensity

13. (c)

Sol. Group index method → Empirical Method Using soil classification test

CBR Method → Empirical Method using soil strength test

US Nowy Method → Semi theoretical Method

Asphalt Institute Method → Qwasi Rational Method

14. (b)

Sol.

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

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

$$\sigma_{tc} = \frac{E\alpha t}{3(1-\mu)}\sqrt{\left(\frac{a}{l}\right)} \quad \text{(iii)}$$

Where α = thermal coefficient of concrete.

t = temp difference between top and bottom

C_x and C_y are the coefficient based on the $\left(\frac{L_x}{l}\right)$ and $\left(\frac{L_y}{l}\right)$

$\mu = 0.15$

Hence (b) option is correct

15.

Sol. As per IRC code No. 37 – 1985 deals with Design of Flexible pavement taking commutative axle loads in msa and CBR into account.

16. (b)

Sol. Radius of Relative Stiffness

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

Hence Radius of Relative Stiffness does not depend upon wheel load.

17.

Sol. At corner and edge location in a cement concrete pavement will be combined stresses due to traffic wheel load and temp have to be critically checked during design.

18. (d)

Sol. Radius of Relative Stiffness and modulus of subgade reaction is related to design of thickness of rigid pavement by Wastergard Method.

19. (d)

Sol. G.I. = 0.2a + 0.005ac + 0.01bd

Where a = (L.L. – 35)

b = (% passing 75 mm sieve – 15)

Hence group index depends upon L.L, plasticity index and percent passing 75 M sieve.

20. (b)

Sol. • Dowels bars are provided at expansion joints and sometimes also at contraction joints in cement concrete slabs.

• Tie bars are provided in longitudinal joints.

21. (c)

Sol. • Tie bars are used in cement concrete slabs across the longitudinal joints.

• In contrast to Dowel bars, Tie bars are not load transfer devices.

22. (c)

Sol. Tie Bars are provided across the longitudinal joints at mid depth of slabs.

23. (c)

Sol. Bradbury Derived the following formulae for the warping stresses

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

$$\sigma_{ti} = \frac{E\alpha\Delta t}{2}\left[\frac{C_x + \mu C_y}{1-\mu^2}\right]$$

$$\sigma_{tc} = \frac{E\alpha\Delta t}{3(1-\mu)}\sqrt{\left(\frac{a}{l}\right)}$$

$$\text{where } l = \left(\frac{Eh^3}{12k(1-\mu^2)}\right)^{\frac{1}{4}}$$

24. (d)

Sol. Where load stress are given by

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

Ge

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

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

$$\text{where } l = \left[\frac{Eh^3}{12k(1-\mu^2)}\right]^{\frac{1}{4}}$$

Hence (d) option is correct.

25. (c)

Sol.

$$\text{CB.R} = \frac{3.5\times100}{\left(\dfrac{1370\times9.81}{\frac{\pi}{4}\times(50)^2}\right)} \quad 51.13\%$$

[∵ dia of Plunges d = 50 mm]

27. (a)

Sol.

$$k_1 a_1 = k_2 a_2$$

$$\Rightarrow k_2 = \left(\frac{k_1 a_1}{a_2}\right)$$

$$= \frac{16\times30}{75} = 6.4 \text{ kg/cm}^3$$

29. (b)

Sol. Vertical compressive stress is maximum on the pavement surface directly under the wheel load.

32. (d)

The combination of wheel load and temperature can be critical at the Edge.

33. (d)

Sol. For one lane

$$N = \frac{365A\left[(1+r)^n - 1\right]\times VDF\times(T.D.F.)}{r\times10^6}$$

$$= \frac{365\times2500\left[(1+0.8)^{10}-1\right]\times3.5\times0.75}{(0.08\times10^6)}$$

$$= 34.69$$

For two lane

$$N = 2 \times 34.69 = 69.399$$
$$= 70 mSA.$$

39.

Sol.

$$N = \frac{365 \times 1213 \times \left[(1+.08)^{12}-1\right] \times 2.5 \times 1}{(.08 \times 10^6)}$$

$$= 21.00 \text{ MSA}.$$

43. (c)

Sol.

$$N = \frac{365A\left[(1+r)^n-1\right] \times V.D.F.}{r \times 10^6}$$

$$= \frac{365 \times 2723 \times \left[(1+.05)^{10}-1\right] \times 2.4}{(0.05 \times 10^6)}$$

$$= 30.00 \text{ MSA}$$

Total thickness of the pavement = 670mm

44. (b)

Sol.

$$Le = \frac{\delta}{2\alpha(T_2-1)}$$

$$= \frac{\cancel{2}010}{\cancel{2} \times 10 \times 10^{-6} \times (60-20)}$$

$$= \frac{10^6 \times 10^{-3}}{40} = 25 \text{ mm}$$

45.

Sol. As per IRC – 37 – 1984

$$N_1 = \frac{365A\left[(1+r)^n-1\right] \times V.D.F. \times T.D.F.}{r \times 10^6}$$

= 51.64 MSA

$$= \frac{365 \times 2000\left[(1+.075)^{10}-1\right] \times 5}{(.075 \times 10^6)}$$

$$= 10.33 \text{ MSA}$$

$$N_2 = \frac{365 \times 200\left[(1+.075)^{10}-1\right] \times 6}{(.075 \times 10^6)}$$

$$= 6.196$$

$$N = N_1 + N_2 = 57.8 \text{ MSA}$$

41. (a)

Sol. Equivalent Daily mo of repetitions for the standard Axle load

$$= \left(\frac{40}{80}\right)^4 \times 800 + \left(\frac{80}{80}\right)^4 \times 400$$

$$= 450$$

40. (b)

Sol. At corner there is no contribution of Frictional stress

$\because$ At corner = 30 + 9 = 39 kg/cm²

At Edge during Summer

= (32 + 8 – 5) = 35 kg/cm²

At Edge during Winter

= (32 + 6 – 45) = 42 kg/cm²

$\because$ The most critical stress = 42 kg/cm²

42. (d)

Sol. We know that load stress is given by

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

$$\sigma_e = \frac{0.752P}{h^2}\left[4\log\left(\frac{l}{b}\right)+0.359\right]$$

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

Hence d option is correct

35. (b)

Sol. Origin and Destination survey is done for Traffic survey.

- Collision diagram is made for Accident studies
- OMC → Compaction
- Radius of Relative Stiffness → Concrete Pavement design.

26. (b)

Sol.

$$G_t = \frac{100}{\frac{(100-6)}{2.65} + \frac{6}{1.09}}$$

$$= 2.4147$$

$\therefore$ Density in kg/m³ of betuminson concrete

$$= 2.4147 \times 1000$$
$$= 2414.7 \text{ kg/m}^3$$

48.

Sol. G.I. = 0.2a + 0.005ac + 0.01bd

= (0.2 × 15 + .005 × 15 + 0.01 × 35 × 10)

= 3 + 3.5 = 6.5

CHAPTER

6

Highway Construction

EMBANKMENT CONSTRUCTION

Materials and General Requirements

- The materials used in embankments, subgrades, earthen shoulders and miscellaneous backfills shall be soil, moorum, gravel, a mixture of these.
- The following types of material shall be considered unsuitable for embankment:

  Clay having liquid limit exceeding 70 and plasticity index exceeding 45; and materials with salts resulting in leaching in the embankment.
- Expansive clay exhibiting marked swell and shrinkage properties ("free swelling index" exceeding 50 per cent shall not be used as a fill material.
- Where an expansive clay with acceptable "free swelling index" value is used as a fill material, subgrade and top 500 mm portion of the embankment just below subgrade shall be non-expansive in nature.
- Ordinarily, only the materials satisfying the density rquirements given in table below shall be employed for the construction of the embankment and the subgrade.

Table-I : Density Requirements of Embankment and Subgrade Materials

S.No.	Type of Work	Maximum laboratory dry unit wt.
1.	Embankments up to 3 metres height, not subjected to extensive flooding.	Not less than 15.2 kN/cu.m.
2.	Embankments exceeding 3 metres height or embankments of any height subject to long periods of inundation	Not less than 16.0 kN/cu.m.
3.	Subgrade and earthen shoulders/verges/backfill	Not less than 17.5 kN/cu.m.

Table-II Compaction Requirements for Embankment and Subgrade

Type of work/material	Relative compaction as per centage of max. laboratory dry density
1. Subgrade and earthen shoulders	Not less than 97

Construction Operations

- **Compacting ground supporting embankment/subgrade:** Where necessary, the original ground shall be levelled to facilitate placement of first layer of embankment, Scarified, mixed with water and then compacted by rolling so as to achieve minimum dry density as given in Table II above.
- In case where the difference between the subgrade level (top of the subgrade on which pavement rests) and gound level is less than 0.5 m and the ground does not have 97 per cent relative compaction with respect to the dry density as given inTable II, the ground shall be loosened upto a level 0.5 m below the subgrade level, watered and compacted in layers to not les than 97 per cent of dry deensity as given in Table II.
- The embankment and subgrade material shall be spread in layers of uniform thickness not exceeding 200 mm by mechanical means, finished by a motor grader and compacted.
- Clods or hard lumps of earth shall be broken to have a maximum size of 75 mm when being placed in the embankment and a maximum size of 50 mm when being placed in the subgrade.

Earthwork for widening existing road embankment :

- When an existing embankment and/or subgrade is to be widened and its slopes are steeper than 1 vertical on 4 horizontal continuous horizontal benches, each at least 300 mm wide, shall be cut into the old slope for ensuring adequate bond with the fresh embankment/subgrade material to be added.
- However, when the existing slope against which the fresh material is to be placed is flatter than 1 vertical on 4 horizontal, the slope surface may only be ploughed or scarified instead of resorting to benching.

Earthwork over existing road surface :

Where the embankment is to be placed over an existing road surface, the work shall be carried out as indicated below:

(i) If the existing road surface is of granular or bituminous type and lies within 1m of the new subgrade level, the same shall be scarified to a depth of 50 mm or more so as to provide ample bond between the old and new material ensuring that at least 500 mm portion below the top of new subgrade level is compacted to the desired density.

(ii) If the existing road surface is of cement concrete type and lies within 1m of the new subgrade level the same shall be removed completely.

(iii) If the level difference between the existing road surface and the new top subgrade level is more than 1m, the existing surface shall be permitted to stay in place without any modification.

Embankment construction under water :

Where filling or backfilling is to be placed under water, only acceptable granular material or rock shall be used.

- Acceptable granular material shall consist of graded, hard durable particles with maximum particle size not exceeding 75 mm.
- The material should be non-plastic having uniformity coefficient of not less than 10.

CONSTRUCTION OF SUB-BASES, BASES (NON-BITUMINOUS) AND SHOULDERS

Granular Sub-Base

- This work shall consist of laying and compacting well-graded material on prepared subgrade.
- The material shall be laid in one or more layers as sub-base or lower sub-base and upper sub-base.
- The material to be used for the work shall be natural sand, moorum, gravel, crushed stone, or combination thereof depending upon the grading required.
- While the gradings in Table-III are in respect of close-graded granular sub-base materials, one each for maximum particle size of 75 mm, 53 mm and 26.5 mm, the corresponding gradings for the coarsegraded materials for each of the three maximum particle sizes are given at Table-IV.
- The water absorption value of the coarse aggregate if is greater than 2 per cent, the soundness test shall be carried out.
- For Grading II and III materials, the CBR shall be determined at the density and moisture content likely to be developed in equilibrium condition, which shall be taken as being the density relating to a uniform air voids content of 5 per cent.

Table-III Grading for Close-Graded Granular Sub-Base Materials

IS Sieve Designation	Per cent by weight passing the IS sieve Grading -I	Grading -II	Grading -III
75.0 mm	100	–	–
53.0 mm	80-100	100	–
26.5 mm	55-90	70-100	100
9.50 mm	35-65	50-80	65-95
4.75 mm	25-55	40-65	50-80
2.36 mm	20-40	30-50	40-65
0.425 mm	10-25	15-25	20-35
0.075 mm	3-10	3-10	3-10
CBR Value (Minimum)	30	25	20

Table-IV Grading for Coarse-Graded Granular Sub-Base Materials

IS Sieve Designation	Per cent by weight passing the IS sieve Grading -I	Grading -II	Grading -III
75.0 mm	100	–	–
53.0 mm		100	
26.5 mm	55-75	50-80	100
9.50 mm			

4.75 mm	10-30	15-35	25-45
2.36 mm			
0.425 mm			
0.075 mm	<10	<10	<10
CBR Value (Minimum)	30	25	20

Note : The material passing 425 micron (0.425 mm) sieve for all the three gradings when tested shall have liquid limit and plasticity index not more than 25 and 6 per cent respectively.

Construction Operations

- **Preparation of subgrade :** Immediately prior to the laying of sub-base, the subgrade already finished shall be prepared by removing all vegetation and other extraneous matter, lightly sprinkled with water if necessary and rolled with two passes of 80-100 kN smooth wheeled roller.
- **Spreading and compacting :** The sub-base material shall be spread on the prepared subgrade with the help of a motor grader.

  Moisture content of the loose material shall be checked and suitably adjusted by sprinkling additional water from a truck mounted or trailer mounted water tank. So that, at the time of compaction, it is from 1 per cent above to 2 per cent below the optimum moisture content.
- Immediately thereafter, rolling shall start.
- If the thickness of the compacted layer does not exceed 100 mm, a smooth wheeled roller of 80 to 100 kN weight may be used.
- For a compacted single layer upto 225 mm the compaction shall be done with the help of a vibratory roller of minimum 80 to 100 kN static weight or hea vy pneumatic tyred roller of minimum 200 to 300 kN weight having a minimum typre pressure of 0.7 mN/m^2.
- Rolling shall commence at the lower edge and proceed towards the upper edge longitudinally for portions having unidirectional crossfall and super elevation and shall commence at the edges and progress towards the centre for portions having crossfall on both sides.
- Each pass of the roller shall uniformly overlap not less than one third of the track made in the preceding pass.
- During rolling, the grade and crossfall (camber) shall be checked and any high spots or depressions, which become apparent, corrected by removing or adding fresh material.
- The speed of the roller shall not exceed 5km per hour.
- Rolling shall be continued till the density achieved is at least 98 per cent of the maximum dry density for the material.

Lime Treated Soil for Improved Sub-Grade/sub-Base

- This work shall consist of laying and compacting an improved sub-grade/lower sub-base of soil treated with lime on prepared sub-grade.
- Lime treatment is generally effective for soils which contain a relatively high per centage of clay and silty clay.

- The soil used for stabilisation shall be the local clayey soil having a plasticity index greater than 8.
- Lime for lime-soil stabilisation work shall be commercial dry lime slaked at site or pre-slaked lime.
- Lime shall be properly stored to avoid prolonged exposure to the atmosphere and consequent carbonation which would reduce its binding properties.
- The mix design shall be done to arrive at the appropriate quantity of lime to be added, having due regard to the purity of lime, the type of soil, the moisture-density relationship, and the design CBR/Unconfined Compressive Strength (UCS) value specified.
- The laboratory CBR/UCS value shall be at least 1.5 times the minimum field value fo CBR/UCS stipulated.
- Lime-soil stabilisation shall not be done when the air temperature in the shade is less than 10°C.
- The thickness of any layer to be stabilised shall be not less than 100 mm when compacted. The maximum thickness shall be 200 mm.
- Lime may be mixed with the prepared material either in slurry form or dry state.
- Dry lime shall be prevented from blowing by adding water to the lime.
- The moisture content at compaction checked shall neither be less than the optimum moisture content nor more than 2 per cent above it.
- Immediately after spreading, grading and levelling of the mixed material, compaction shall be carried out.
- Rolling shall commence at edges and progress towards the centre, except at superelevated portions where it shall commence at the inner edge and progress towardes outer edge.
- Compaction shall continue until the density achieved is at least 98 per cent of the maximum dry density for the material.
- Care shall be taken to see that the compaction of lime stabilised material is completed within three hours of its maxing.
- The sub-base course shall be suitably cured for a minimum period of 7 days after which subsequent pavement courses shall be laid to prevent the surface from drying out and becoming friable.
- When lime is used for improving the subgrade, the soil-lime mix shall be tested for its CBR value.
- When lime stabilised soil is used in a sub-base, it shall be tested for unconfined compressive strength (UCS) at 7 days.

Cement Treated Soil Sub-Base/Base

- The material used for cement treatment shall be soil including sand and gravel, laterite, kankar, brick aggregate, crushed rock or slag or any combinatin of these.
- For use in a sub-base course, the material shall have a grading shown in Table-IV.
- It shall have a uniformity coefficient not less than 5, capable of producing a well closed surface finish.
- For use in a base course, the materials shall be sufficiently well graded to ensure a well-closed surface finish and have a grading within the range given in Table-IV.

- If the material passing 425 micron sieve is plastic, it shall have a liquid limit not greater than 45 per cent and a plasticity index not greater than 20 per cent determined in accordance with IS:2720 (Part 5).

Construction Operations

- Stabilisation shall not be done when the air temperature in the shade is less than 10°C.
- For stabilisation, the soil before addition of stabilizer, shall he pulverised, where necessary, to the extent that it passes the requirements as set out in Tahle-V.

Table-V : Soil Pulverisation Requirements for Cement Stabilisation

IS Sieve designation	Minimum per cent by weight passing the IS sieve
26.5 mm	100
5.6 mm	80

- The moisture content at compaction shall not be less than the optimum moisture content nor more than 2 per cent above it
- The compaction of cement stabilised material is completed within two hours of its mixing.
- The sub-base/base course shall be suitably cured for 7 days. Subsequent pavement course shall be laid soon after to prevent the surface from drying out and becoming friable.

Water Bound Macadam Sub-Base/Base

- This work shall consist of clean, crushed aggregates mechanically interlocked by rolling and bonding together with screening, binding material where necessary and water laid on a properly prepared subgrade/sub-base/base or existing pavement, as the case may be and finished.
- It is not desirable to lay water bound macadam on an existing thin black topped surface without providing adequate drainage, facility for water that would get accumulated at the interface of existing bituminous surface and water bound macadam.

Materials

Coarse aggregates :

- Coarse aggregates shall be either crushed or broken stone, crushed slag, overburnt (Jhama) brick aggregates or any other naturally occuring aggregates such as kankar and laterite of suitable quality.
- Materials other than crushed or broken stone and crushed slag shall be used in sub-base courses only.
- If crushed gravel/shingle is used, not less than 90 per cent by weight of the gravel/shingle pieces retained on 4.75 mm sieve shall have at least two fractured faces.
- The aggregates shall conform to the physical requirements set forth in Table-VI.
- If the water absorption value of the coarse aggregate is greater than 2 per cent, the soundness test shall be carried out on the material.
- **Crushed or broken stone :** The crushed or broken stone shall be hard, durable and free from excess flat, elongated, soft and disintegrated particles, dirt and other deleterious material

Crushed slag : Crushed slag shall be made from air-cooled blast furnace slag. The weight of crushed slag shall not be less than 112 kN per m^3 and the per centage of glossy material shall not be more than 20.

- **Overburnt (Jbam) brick aggregates :** Jhama brick aggregates shall be made from overburnt bricks or brick bats and be free from dust and other objectionable and deleterious materials.
- **Grading requirement of coarse aggregates :** The coarse aggregates shall conform to one of the Gradings given in Table- VII as specified, provided, however, the use of Grading No.1 shall be restricted to sub-base courses only.

Table-VII Grading Requirements of Coarse Aggregates

Grading No.	Size Range	IS Sieve Designation	Per cent by weight passing
1.	90 mm to 45 mm	125 mm	100
		90 mm	90-100
		63 mm	25-60
		45 mm	0-15
		22.4 mm	0-5
2.	63 mm to 45 mm	90 mm	100
		63 mm	90-100
		53 mm	25-75
		45 mm	0-15
		22.4 mm	0-5
3.	53 mm to 22.4 mm	63 mm	100
		53 mm	95-100
		45 mm	65-90
		22.4 mm	0-10
		11.2 mm	0-5

Note : The compacted thickness for a layer with Grading 1 shall be 100 mm while for layer with other Gradings i.e. 2 and 3. it shall be 75 mm

Screenings: Screenings to fill voids in the coarse aggregate shall generally consist of the same material as the coarse aggregate.

- However, where permitted, predeominantly non-plastic material such as moorum or gravel (other than rounded river borne material) may be used for this purpose provided liquid limit and plasticity index of such material are below 20 and 6 respectively and fraction passing 75 micron sieve does not exceed 10 per cent.

Screenings shall conform to the grading set forth in Table-VIII.

- The use of screenings shall be omitted in the case of soft aggregates such as brick metal, kankar, laterites, etc. as they are likely to get crushed to a certain extent under rollers.

Table-VIII : Grading for Screenings

Grading Classification	Size of Screenings	IS Sieve Designation	Per cent by weight passing the IS Sieve
A	13.2 mm	13.2 mm	100
		11.2 mm	95-100
		5.6	15-35
		180 micron	0-10
B	11.2 mm	11.2 mm	100
		5.6 mm	90-100
		180 micron	15-35

Binding material : Binding material to be used for water bound macadam as a filler material is meant for preventing ravelling, shall have a Plasticity Index (PI) value of less than 6.

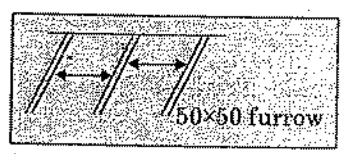
- Application of binding materials may not be necessary when the screenings used are of crushable type such as moorum of gravel.

Construction Operations

Preparation of base : The surface of the subgrade/ sub-base/base to receive the water bound macadam course shall be prepared to the specified lines and crossfall (camber) and made free of dust and other extraneous material.

- Any sub-base/base/surface irregularities, where predominant, shall be made good by providing approapriate type of profile corrective course.
- As far as possible, laying water bound macadam course over an existing thick bituminous layer may be avoided since it will cause problems of internal drainage of the pavement at the interface of two courses.
- It is desirable to completely pick out the existing thin bituminous wearing course where water bound macadam is proposed to be laid over it.
- However, where the intensity of rain is low and the interface drainage facility is efficient, water bound macadam can be laid over the existing thin bituminous surface by cutting 50 mm × 50 mm furrows at an angle of 45 degrees to the centre line of the pavement at one metre intervals in the existing road.

  The directions and depth of furrows shall be such that they provide adequate bondage and also serve to drain water to the existing granular base course beneath the existing thin bituminous surface.



- If water bound macadam is to be laid directly over the subgrade, without any other intervening pavement course, a 25 mm course of screenings (Grading B) or coarse sand shall be spread on the prepared subgrade before application of the aggregates is taken up.
- In case of a fine sand or silty or clyey subgrade, it is advisable to lay 100 mm insulating layer of screening or coarse sand on top of fine grained soil, the gradation of which will depend upon whether it is intended to act as a drainage layer as well.
- Sometimes approapriate geosynthetics, performing functions of separation and drainage, may be used over the prepared subgrade.

Spreading coarse aggregates : The coarse aggregates shall be spread uniformly and evenly upon the prepared subgrade/sub-base/ base to proper profile by using templates placed across the road about 6m apart, in such quantities that the thickness of each compacted layer is not more than 100 mm for Grading 1 and 75 mm for Grading 2 and 3.

Rolling : Immediately following the spreading of the coarse aggregate, rolling shall be started with three wheeled power rollers of 80 to 100 kN capacity or tandem or vibratory rollers of 80 to 100 kN static weight.

- Except on superelevated portions where the rolling shall proceed from inner edge to the outer, rolling shall begin from the edges gradually progressing towards the centre.
- First the edge/edges shall be compacted with roller running forward and backward. The roller shall then move inward parallel to the centre line of the road, in successive passes uniformly lapping preceding tracks by at least one half width.
- Rolling shall be discontinued when the aggregates are partially compacted with sufficient void space in them to permit application of screenings.
- However, where screeings are not to the applied, as in the case of crushed aggregates like brick metal, laterite and kankar, compaction shall be continued until the aggregates are thoroughly keyed.
- During rolling, slight sprinkling of water may be done, if necessary.
- The rolled surface shall be checked transversely and logitudinally, with templates and any irregularities corrected by loosening the surface, adding or removing necessary amount of aggregates and re-rolling until the entire surface conforms to desired crossfall (camber) and grade.
- In no case shall the use of screenings be permitted to make up depressions.
- It shall be ensured that shoulders are built up simultaneously along with water bound macadam courses .

Application of screenings :

After the coarse aggregate has been rolled screenings to completely fill the interstices shall be applied gradually over the surface.

- These shall not be damp or wet at the time of applicatin.
- Dry rolling shall be done while the screenings are being spread so that vibrations of the roller cause them to settle into the voids of the coarse aggregate.
- The screeings shall be applied at a slow and uniform rate (in three or more applications) so as to ensure filling of all voids. This shall be accompanied by dry rolling and brooming with mechanical brooms, hand-brooms or both.
- In no case shall the screenings be applied so fast and thick as to form cakes or ridges on the surface in such a manner as would prevent filling of voids or prevent the direct bearing of the roller on the coarese aggregate.
- These operations shall continue until no more screeings can be forced into the voids of the coarse aggregate.
- The spreading, rolling, and brooming of screenings shall be carried out in only such lengths of the road which could be completed within one dya's operation.

Sprinkling of water and grouting :

After the screenings have been applied, the surface shall be used to sweep the wet screenings into voids and to distribute them evenly.

- The sprinkling, sweeping and rolling operation shall be continued, with additional screenings applied as necessary until the coarse aggregate has been throughly keyed, well-bonded and firmly set in its full depth and a grout has been formed of screenings.
- In case of lime treated soil sub-base, construction of water bound macadam on top of it can cause excessive water to flow down to the lime treated sub-base before it has picked up enough strenth (is still green) and thus cause damage to the sub-base layer.
- The laying of water bound macadam layer in such cases shall be done after the sub-base attains adequate strength.

Application of binding material : After the application of screenings, the binding material where it is required to be used shall be applied successively in two or more thin layers at a slow and unifrom rate.

- After each application, the surface shall be copiously sprinkled with water, the resulting slurry swept in with hand brooms, or mechanical brooms to fill the voids properly, and rolled during which water shall be applied to the wheels of the rollers if necessary to wash down the binding material sticking to them.
- These operations shall continue until the resulting slurry after filling of voids, forms a wave ahead of the wheels of the moving roller.

Setting and drying : After the final compaction of water bound macadam course, the pavement shall be allowed to dry overnight. Next morning hungry spots ahll be filled with screenings or binding material lightly sprinkled with water if necessary and rolled.

- No traffic shall be allowed on the road until the macadam has set.
- The compacted water bound macadam course should be allowed to completely dry and set before the next pavement course is laid over it

Wet Mix Macadam Sub-base

This work shall consist of laying and compacting clean, crushed, graded aggregate and granular material, premixed with water, to a dense mass on a prepared subgrade/sub-base/base or existing pavement.

- The material shall be laid in one or more layers as necessary to lines, grades and cross-sections.
- The thickness of a single compacted Wet Mix Macadam layer shall not be less than 75 mm.
- When vibrating or other approved types of compacting equipment are used, the compacted depth of a single layer of the sub-base course may be increased to 200 mm.

Materials

Aggregates

Physical requirements : Coarse aggregates shall be crushed stone.

- If crushed gravel/shingle is used, not less than 90 per cent by weight of the gravel/shingle pieces retained on 4.75 mm sieve shall have at least two fractured faces. The aggregates shall conform to the physical requirements set forth in Table X below.

Table-X Physical Requirements of Coarse Aggregates for Wet Mix Macadam for Sub-Base/Base Course

Test	Test Method	Requirements
1. *Los Angeles Abrasion value	IS : 2386 (Part-4)	40 per cent (Max)
or		
*Agregate Impact value	IS : 2386 (Part-4) or IS : 5640	30 per cent (Max)
2. Combined Flakiness and Elongation indices (Total)	IS : 2386 (Part-1)	30 per cent (Max)**

* Aggregate may satisfy requirements of either of the two tests.

** To determine this combined proportion, the flaky stone from a representative sample should first be separated out. Flakiness index is weight of flaky stone metal divided by weight of stone sample. Only the eleongated particles be separated out from the remaining (non-flaky) stone metal. Elongation index is weight of elongated particles divided by total non flaky particles. The value of flakiness index and elongation index so found are added up.

- If the water absorption value of the coarse aggregate is greater than 2 per cent, the soundness test shall be carried.

Grading requirements : The aggregates shall conform to the grading given in Table-11.

Table-11: Grading Requirements of Aggregates for Wet Mix Macadam

Is Sieve Designation	Per cent by weight passing the IS sieve
53.00 mm	100
45.00 mm	95.100
26.50 mm	—
22.40 mm	60-80
11.20 mm	40-60
4.75 mm	25-40
2.36 mm	15-30
600.00 micron	8-22
75.00 micron	0-8

- Materials finer than 425 micron shall have Plasticity Index (PI) not exceeding 6.

Construction Operations

Preparation of base : Same as in WBM

Provision of lateral confinement of aggregates : While constructing wet mix macadam, arangement shall be mode for the lateral confinement of wet mix.

- This shall be done by laying materials in adjoining shoulders along with that of wet mix macadam layer.

Preparation of mix : Wet Mix Macadam shall be prepared in an approved mixing plant of suitable capacity having provision for controlled addition of water and forced/positive mixing arrangement like pugmill or pan type mixer of concrete batching plant.

Spreading of mix : Immediately after mixing, the aggregates shall be spread uniformly and evenly upon the prepared subgrade/sub-base/base in required quantities.

Compacton : After the mix has been laid to the required thickness, grade and crossfall/camber the same shall be uniformly compacted, to the full depth with suitahle roller.

- If the thickness of single compacted layer does not exceed 100 mm, a smooth wheel roller of 80 to 100 kN weight may be used.
- For a compacted single layer upto 200 mm, the compaction shall be done with the help of vibratory roooler of minimum static weight of 80 to 100 kN.
- The speed of the roller shall not exceed 5 km/h.
- Rolling should not be done when the subgrade is soft or yielding or when it causes a wave-like motion in the sub-base/base course of subgrade.
- If irregularities develop during rolling which exceed 12 mm when tested with a 3 metre straight edge, the surface should be loosened and premixed material added or removed as required before rolling again so as to achieve a unifrom surface conforming to the desired grade and crossfall.
- In no case should the use of unmixed material be permitted to make up the depressions.
- Rolling shall be continued till the density achieved is at least 98 per cent of the maximum dry density for the material.

Setting and drying : After final compactino of wet mix macadam course, the road shall be allowed to dry for 24 hours.

- Preferably no vehicular traffic of any kind should be allowed on the finished wet mix macadam surface till it has dried and the wearing course laid.

BASE AND SURFACE COURSES (BITUMINOUS)

Preparation of Surface

- This work shall consist of preparing an existing granular or black-topped surface to specified lines, grades and cross-section in advance of laying a bitumminous course.
- The work shall consist of scarifying and re-laying the granular base course and/or scarifying the existing surface, filling of potholes, sealing of cracks and/or application of a profile corrective course (levelling course) as necessary.

Materials

For scarifying and re-laying the granular surface : The materials used shall be coarse aggregates salvaged from scarification of the existing granular base course supplemented by fresh coarse aggregates and screenings so that aggregates and screenings thus supplemented correspond to Water Bound Macadam.

For patching potholes & sealing cracks : For patching potholes, approved material having same specification as that of profile corrective course shall be used.

- For sealing small cracks finer than 3 mm, a fog seal shall be applied while larger cracks wider than 3 mm shall be treated with an emulsion slurry seal.
- **For profile corrective course :** A profile corrective course (levelling course) is essentially a pavement base material course for correcting the existing pavement profile which has either lost its shape or has to be given a new shape to meet the requirement of specified lines, grades and cross-sections.
- It shall be differentiated from the strengthening course or other type of structural pavement course needed for upgrading as a remedial measure against inherently deficient and/or distressed pavement.
- It is meant to remove the irregularity in the existing road profile only.

Profiel corrective course and its application : If it is to be laid as part of the overlay/strengthening course, the profile corrective course material shall be of the same specification as that of the overlay/strengthening course.

- However, if provided as a separate layer, it may be of the same specification as the layer over which it is to be laid or intermediate between underlying and overlying layers.

Construction Operations

Preparing existing granular surface : Where the existing surface is granular, all loose and disintegrated materials shall be removed and the surface lightly watered if the profile corrective course to be provided as a separeate layer is also granular.

- If, however, over the existing granular surface, a profile corrective course of bituminous material is to be laid, the existing granular surface shall be primed.

Scarifying existing bituminous surface : Where necessary, the existing bituminous layer in the specified width shall be removed with care without causing undue disturbance to the underlying layer.

- After removng it, all loose and disintegrated materials of underlying layer which might have been disturbed in the process of removal shall, before laying of the overlay course, be reset properly by spreading/hand packing of aggregates and compacting with suitable roller/heavy hand rammers/approved mechanical tamper so that the level of the top surface of such scarified area shall be even and properly graded with respect to adjoining surface.
- Where applicable, the granular surface, after removal of the existing bituminous layer, shall be primed to receive a bituminous profile corrective course.

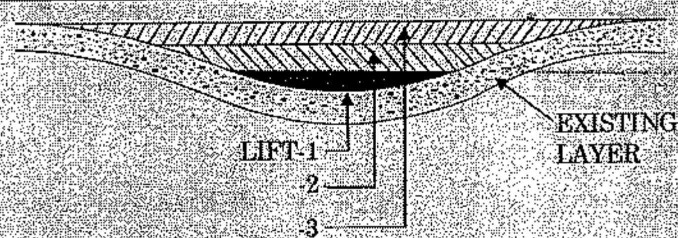
Patching of potholes and sealing of cracks : Before providing profiel corrective course on the existing pavement, potholes, if any, shall be drained of water, cut to regular shape with sides vertical upto the affected depth and slightly beyond the limits of affected area and dried.

- All loose and disintegrated materials from it shall be removed. The potholes shall then be filled with material in layers not exceeding 75 mm after painting the sides and bottom with a thin layer of hot straight-run bitumen/emulsion and each layer shall be compacted with approved mechanical tampers/ small vibratory roller and the top layer shall be flush with the existing bituminous surface.

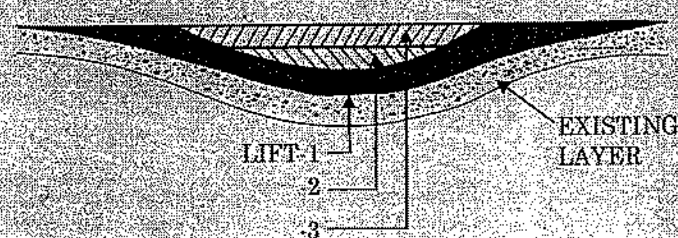
- The cracks in the old pavement surface shall be sealed with a fog seal if cracks are small (less than 3mm width).
- Fog seal shall consist of a spray of a bituminous cutback or a slow-setting bitumen emulsion diluted with an equal amount of water.
- The rate of spray being 0.5 to 1.0 litre/sq.m depending upon the texture and dryness of the existing bituminous surface.
- The spray is allowed to set to a firm condition and traffic is allowed only thereafter so as to ensure that the material is not picked up by traffic.
- For large cracks, the sealing shall be done with emulsion slurry seal.

Laying the profile correct course

- After preparing the granular surface, the profile corrective course shall be laid and compacted. Where a bituminous profile corective course is to be laid over a primed granular surface, a tack coat shall be applied prior to laying profile corrective course.
- An existing bituminous surface shall be prepared and after applying a tack coat the bituminous profile corrective course shall be laid and compacted.
- For camber correction or correction of superelevation of the existing carriageway, method as shown in the illustrative fig. shall be adopted depending on the profile of the existing carriageway.

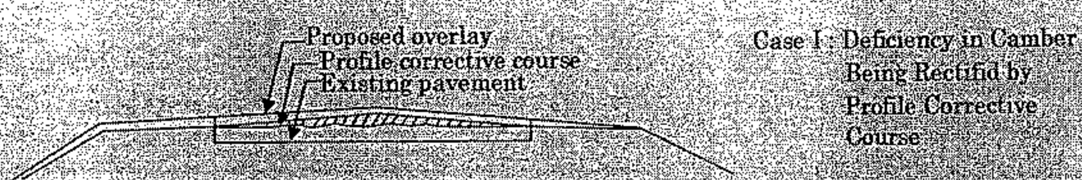


(A) CORRECT METHOD



(B) INCORRECT METHOD

Methods for providing corrective course for short sags and depressions



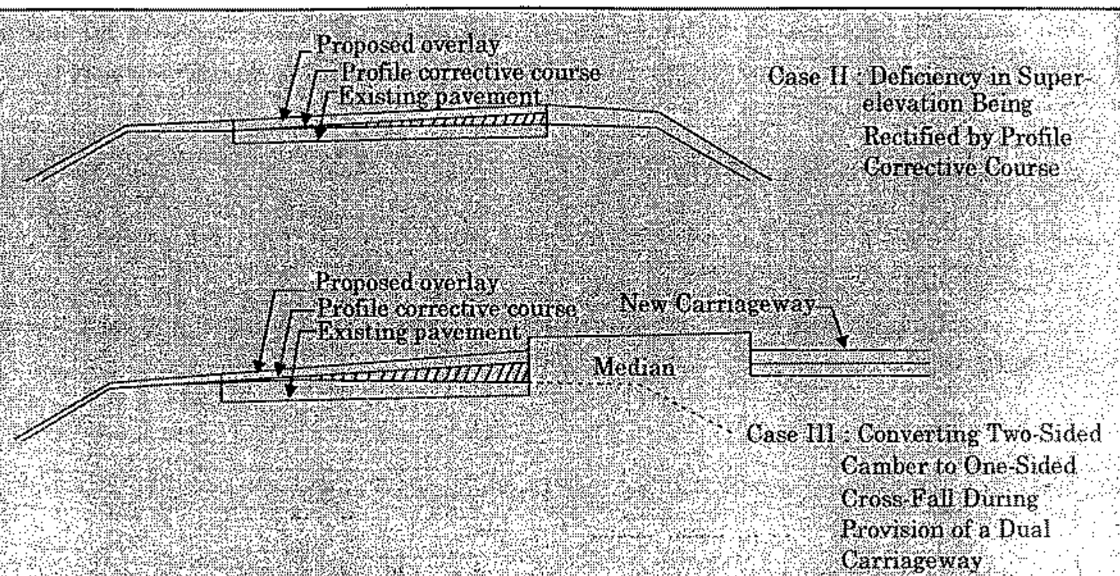


Fig. Correction of Camber or Super-Elevation

Covering the profile corrective course : Work of Profile Corrective Course shall be so planned that it shall be covered by the designed base/wearing course at the earliest, before opening to regular traffic.

Prime Coat Over Granular Base

- This work shall consist of application of single coat of low viscosity liquid bituminous material to an absorbent granular surface preparatory to any superimposed bituminous treatment or construction. Objective is to plug in the capillary voids of the porous surface and to bond the loose material particles on existing surface using a binder of low viscosity which can penetrate into voids.

Materials

- The choice of a bituminous primer shall depend upon the porosity characteristics of the surface to be primed.

  These are :

  (i) Surfaces of low porosity; such as wet mix macadam and water bound macadam,

  (ii) Surfaces of medium porosity; such as cement stabilized soil base,

  (iii) Surfaces of high porosity; such as a gravel base.

Table Viscosity Requirement and Quantity of Bituminous Primer

Type of surface	Kinematic Viscosity of Primer at 60°C (Centistokes)	Quantity per 10 sq.m. (kg)
Low porosity	30-60	6 to 9
Medium porosity	70-140	9 to 12
High porosity	250-500	12 to 15

- The bituminous primer shall be Medium Curing Cutback (MC) produced by fluxing, in an approved manner, bitumen of 80/100 penetration grade with kerosene. Slow setting Cationic emulsion may also be used.

Weather and Seasonal Limitations

- The bituminous primer shall not be applied on a wet surface during dust storm or when the weather is foggy, rainy or windy.
- The prime coat for surface treatment should not be applied when the temperature in the shade is less than 10°C.

Construction

- **Preparation of road surface :** The surface to be primed shall be swept clean, free from dust and shall be dry.
- **Application of bituminous primer :** The bituminous primer shall be sprayed/distributed uniformly over the dry surface using self-propelled sprayer equipped with self-heating arrangement, suitable pump, adequate capacity compressor and spraying bar with nozzles having constant volume or pressure system capable of supplying primer at specifed rates and temperature so as to provide a uniformly unbroken spread of primer.
- The primer coat shall be applied only on the topmost water bound macadam or any granular layer, over which the bituminous base course/wearing course is to be laid.

Tack Coat :

- This work shall consist of application of a single coat of low viscosity liquid bituminous material to an existing road surface which is relatively impervious preparatory to another bituminous construction over it.

Materials

Binder : The binder used for tack coat shall be a bituminous emulsion or cutback.

Construction Operation

Preparation of base : The surface on which the tack coat is to be applied shall be cleaned of dust and any extraneous material before the application of the binder.

Table : Rate of Application of Tack Coat

Type Surface	Quantity of liquid Bituminous material in kg per 10 sq. m. are
(i) Normal bituminous surface	2.0 to 2.5
(ii) Dry and hungry bituminous surfaces	2.5 to 3.0
(iii) Granular surfaces treated with primer	2.5 to 3.0
(iv) Non bituminous surfaces	
(a) Granular base (not primed)	3.5 to 4.0
(b) Cement concrete pavement	3.0 to 3.5

Bituminous Macadam

The work shall consist of construction, in a single course, of compacted crushed aggregates premixed with a bituminous binder, to serve as base/binder course, laid immediately after mixing, on a base prepared previously.

Materials

Bitumen : The bitumen shall be paving bitumen of suitable penetration grade (30/40 to 80/100). The actual grade of bitumen to be used shall be appropriate to the region, traffic, rainfall and other environmental conditions.

Aggregates

The aggregates shall consist of crushed stone, crushed gravel/shingle or other stones. They shall be clean, strong, durable, of fairly cubical shape and free from disintegrated pieces.

- The aggregates shall preferably be hydrophobic and of low porosity.
- If hydrophilic aggregates are to be used, the bitumen shall preferably be treated with anti-stripping agents. The aggregates shall satisfy the physical requirements set forth in Table- XI.

Table-XI Physical Requirements of Aggregates for Bituminous Macadam

S.No.	Test	Test Method	Requirement
1.	Los Angeles Abrasion Value*	IS : 2386 (Part-4)	40 per cent Maximum
2.	Aggregate Impact Value*	IS : 2386 (Part-4)	40 per cent Maximum
3.	Flakiness and Elongation** Indices (Total)	IS : 2386 (Part-4)	40 per cent Maximum
4.	Coating and Stripping of Bitumen Aggregate Mixture	AASHTO T 182	Minimum retained coating 95 per cent
5.	Soundness	IS : 2386 (Part-5)	
	(i) Maximum	Loss with Sodium Sulphate 5 cycles	12 per cent
	(ii) Maximum Sulphate 5 cycles	Loss with Magnesium	18 per cent
6.	Water absorption	IS : 2386 (Part-3)	2 per cent Maximum

* Aggregates may satisfy requirements of either of the two tests.

** To determine this cobined proportion, the flaky stone from a representative sample should first be separated out. Flakiness index is weight of flaky stone metal divided by weight of stone sample. Only the elongated particles be separated out from the remaining (non-flaky) stone metal. Elongation index is weight of elongated particles divided by total non-flaky particles. The value of flakiness index and elongation index so found are added up.

Proportioning of materials :

The bitumen content for premixing shall be 3 to 3.5 per cent by weight of the total mix

Table-XII Aggregate Grading for Bituminous Macadam

IS Sieve Designation	Per cent by weight passing the sieve	
	Grading-1	Grading-2
45.0 mm	100	–
26.5 mm	75-100	100
22.4 mm	60-95	75-100
11.2 mm	30-55	50-85
5.6 mm	15-358	20-40
2.8 mm	5-20	5-20
90.0 micron	0-5	0-5

- The maximum compacted thickness of a layer shall be 100 mm.

Construction Operations

Weather and seasonal limitations : The work of laying shall not be taken up during rainy or foggy weather or when the base course is damp or wet, or during dust storm or when the atmospheric temperature in shade is 10°C or less.

Preparation of the base : Bituminous macadam mix shall be prepared in a hot mix plant of adequate capacity and capable of yielding a mix of proper and uniform quality with thoroughly coated aggregates.

- The temperature of binder at the time of mixing shall be in the range of 150°C to 163°C and that of the aggregate in the range of 155°C - 163°C. Provided that the difference in temperature between the binder and aggregate at no time exceeds 14°C.
- Mixing shall be thorough to ensure that a homogeneous mixture is obtained in which all particles of the aggregates are coated uniformly, and the discharge temperature of mix shall be between 130°C to 160° C.

Spreading : The mix transferred from the tipper at site to the paver shall be spread immediately by means of self-propelled mechanical paver with suitable screeds capable of spreading, tamping and finishing the mix true to the specified lines, grades and cross-sections.

- The temperature of the mix at the time of laying shall be in the range of 120°C to 160°C.

Compaction : After the spreading of mix, rolling shall be done by 80 to 100 kN rollers. Rolling shall start as soon as possible after the material has been spread deploying a set of rollers as the rolling is to be completed in limited time frame.

- The roller shall move at a speed not more than 5 km/h.
- The rolling shall commence at the edges and progress towards the centre longitudinally except that on superelevated and uni-directional cambered portions, it shall progress from the lower to the upper edge parallel to the centre line of the pavement.

- The rolling shall then be continued till the entire surface has been rolled to 95 per cent of the average laboratory density (obtained from Marshall specimens compacted), there is no crushing of aggregates and all roller marks have been eliminated.
- Each pass of the roller shall uniformly overlap not less than one-third of the track made in the preceding pass.
- The roller wheel shall be kept damp if necessary to avoid bituminous material from sticking to the wheels and being picked up.
- Rolling operations shall be complete in every respect before the temperature of the mix falls below 100°C.
- The bituminous macadam shall be covered with either the next pavement course or wearing course, as the case may be, without any delay.
- If there is to be any delay, the course shall be covered by a seal coat before allowing any traffic over it.

Bituminous Penetration Macadam

- The work shall consist of construction of one or more layers of compacted crushed coarse aggregates with alternate applications of bituminous binder and key aggregates.
- Thickness of an individual course shall not exceed 75 mm.

Materials

Bitumen : The binder shall be paving bitumen of suitable penetration grade within the range of S-35 to S-90 or A-35 to A-90 (30/40 to 80/100).

Preparation of the base : The base on which the Penetration macadam course is to be laid shall be prepared, shaped and conditioned to the specified lines grades.

- A priming coat where needed shall be applied over the base. A tack coat shall be applied as per procedure for tack coat application.

Table- Grading Requirements of Coarse Aggregates and Key Aggregates for Bituminous Penetration Macadam

	Per cent by weight passing the Sieve			
	For 50 mm compacted thickness		For 75 mm compacted thickness	
IS Sieve Designation	Coarse Aggregate	Key Aggregate	Coarse Aggregate	Key Aggregate
63 mm	–	–	100	–
53 mm	–	–	–	–
45 mm	100	–	58-82	–
26.5 mm	37-72	–	–	100
22.4 mm	–	100	5-27	50-75
13.2 mm	2-20	50-75	–	–
11.2 mm	–	–	–	5-25
5.6 mm	–	5-25	–	–
2.8 mm	0-5	0-5	0-5	0-5

Table- Quantities of Materials Required for 10 sq.m. of Road Surface for Bituminous Penetration Macadam Base/Binder Course

Compacted	*Binder Straight run bitumen*	*Coarse Aggregate*	*Key Aggregate*
50 mm	50 kg	0.60 cu.m.	0.15 cu. m.
75 mm	68 kg	0.90 cu.m.	0.18 cu. m.

Spreading and compacting coarse aggregates :

- The coars aggregate in a dry and clean form shall be spread uniformly and evenly with the help of a self-propelled or tipper tail mounted aggregate spreader capable of spreading aggregate uniformly at the specified rates over the required widths.
- The surface of the layer shall be carefully checked with camber templates and all high and low spots remedied by removing or adding aggregates as may be required.
- The spreading shall be carried no further in advance of the rolling and penetrating operations than can be completed in one average day's work.
- Segregated aggregates or aggregates mixed with earth or other foreign substances shall be removed and replaced with graded aggregates.

Compaction : After the spreading of coarse aggregates, dry rolling shall be done by 80-100 kN smooth-wheeled steel roller. Rolling shall start as soon as possible after the material has been spread.

- The rolling shall commence at the edges, parallel to the centre line of the road and progress towards the centre longitudinally except on superelevated and uni-directional cambered portions, where it shall progress from the lower to the upper edge after the edges have been rolled.
- For superelevated portions, the overlapping should be approximately one-third of the width of the rear wheel on each trip.
- After initial dry roling, the surface shall be checked with a crown template and a 3 metre straight-edge.
- The surface shall not vary more than 10 mm from the template or straight-edge.
- All surface irregularities exceeding the above limit shall be corrected by removing or adding aggregate as required.
- The rolling shall be done until the compacted coarse aggregate has firm surface and that it has a texture that will allow free and uniform penetration of the bituminous material.

Application of bituminous materials :

- After the coarse aggregate has been rolled and checked, the bituminous binder shall be applied at specified temperatures.
- At the time of applying the binder, the aggregates shall be surface dry for full depth of the layer.
- The bituminous material shall preferably be applied by a pressure distributor uniformly over the surface at the specified rate.
- Over small areas, where the use of a spray bar is impracticable, the bituminous material shall be applied by the nozzle attachement

Application of key aggregates : Immediately after the first penetration of bitumen, the key aggregates in a clean and dry state shall be spread uniformly over the surface by means of a mechanical spreader.

- If necessary, the surface shall be broomed to ensure uniform application of the key aggregates.
- The entire surface shall then be rolled with a 80-100 kN smooth steel wheel roller (along with vibratory roller if available).
- The Penetration Macadam shall be provided with final surfacing (binder/wearing course) without any delay.
- If there is to be any delay, the course shall be covered by a seal coat.

Built-Up Spray Grout

- This work shall consist of a two-layer composite construction of compacted crushed coarse aggregates with application of bituminous binder after each layer and key aggregates on top for the second layer.
- Thickness of the course shall be 75 mm.
- Built-up spray grout shall be used in a single course in a pavement structure.

Construction Operations

Preparation of base : The base on which the built-up spray grout course is to be laid shall be Prepared, shaped and conditioned to the specified lines, grades and cross-sections.

- A priming coat where needed shall be applied .

Tack coat : A tack coat over the base shall be applied.

Spreading and rolling coarse aggregates for the first layer : Immediately after the application of tack coat, the coarse aggregates in a dry and clean form shall be spread uniformly and evenly preferably by mechanical means at the rate of 0.5 cu.m. per 10 sq.m. area.

- The surface of the layer shall be carefully checked with templates and all high and low spots remedied by removing or adding aggregates as may be required.
- Immediately after spreading of the aggregates, the entire surface shall be rolled with a 80-100 kN smooth-wheeled roller.
- Rolling shall commence at the edges and progress towards the centre except in superelevated and uni-directional cambered portions where it shall proceed from the lower edge to the higher edge.
- Each pass of the roller shall uniformly overlap not less than one-third of the track made in the preceding pass.
- After initial rolling, the surface shall be checked transversely and longitudinally with templates and any irregularities corrected by loosening the surface, adding or removing necessary amounts of aggregate, followed by rolling.
- Rolling shall be stopped before voids in the aggregate layer are closed to such an extent as to prevent free and uniform penetraton of the binder.

Application of binder-first spray : The binder shall be heated to the temperature appropriate to grade of bitumen and sprayed on aggregate layer at the rate of 15 kg/10 sq.m. (in terms of straight-run bitumen) in a uniform manner with the help of mechanical sprayers capable of spraying bitumen uniformly at specified rates and temperatures.

- Excessive deposits of binder caused by stopping or starting of the sprayers or through leakage or any other reason shall be corrected promptly.

Spreading and rolling of coarse aggregate for tbe second layer : Immediately after the first application of the binder, the second layer of coarse aggregates shall be spread and rolled.

Application of binder-second spray : The second aggregate layer shall then be given a binder spray at the rate of 15 kg/10 sq. m.

Application of key aggregate : Immediately after second application of the binder, key aggregates in a clean and dry state shall be spread uniformly and evenly, preferably by mechanical means at the rate of 0.13 cu.m./10 sq. m. so as to cover the surface completely.

- If necessary, the surface shall be broomed to ensure uniform application of the key aggregates.
- The entire surface shall then be rolled with a 80-100 kN smooth-wheeled roller.
- While rolling is in progress, additional key aggregates, where required, shall be spread by hand.
- Rolling shall continue until the entire course is thoroughly compacted and the key aggregates are firmly in position.
- The built-up-spray-grout shall be provided with final surfacing without any delay.
- If there is to be any delay, the course shall be coverted by a seal coat.

Dense Bituminous Macadam

- This work shall consist of construction in a single course of 50 to 100 mm thick base/binder course to the following specifications on a previously prepared base.

Materials

Bitumen : The bitumen shall be paving bitumen of Penetration Grade S 65 or A 65 (60/70). In case of non-availability of bitumen of this grade, S 90 (80/100) grade bitumen may be used.

Coarse aggregates : The coarse aggregates shall consist of crushed stone, crushed gravel/shingle or other stones.

- The coarese aggregates shall preferably be hydrophobic and of low porosity.
- If hydrophilic aggregates are to be used, the bitumen shall be treated with antistripping agents.

Table- Physical Requirements of Aggregates for Dense Bituminous Macadam

S.No.	Test	Test Method	Requirement
1.	Los Angles Abrasion Value*	IS : 2386 (Part-4)	50 per cent maximum
2.	Aggregate Impact value*	IS : 2386 (Part-4)	30 per cent maximum
3.	Flakiness and Elongation** Indices (Total)	IS : 2386 (Part-1)	30 per cent maximum
4.	Coating and Stripping of Bitumen Aggregate Mixtures	AASHTO T 182	Minimum retained coating 95 per cent
5.	Soundness	IS : 2386 (Part-5)	
	(i) Loss with Sodium	5 cycles Sulphate	12 per cent Maximum
	(ii) Loss with Magnesium	5 cycles Sulphate	18 per cent Maximum
6.	Water absorption	IS : 2386 (Part-3)	2 per cent Maximum

* Aggregates may satisfy requirements of either of the two tests.

** To determine this combined proportion, the flaky stone from a representative sample should first be separated out. Flakiness index is weight of flaky stone metal divided by weight of stone sample. Only the elongated particles be separated out from the remaining (non-flaky) stone metal. Elongation index is weight of elongated particles divided by total non-flaky particles. The value of flakiness index and elongation index so found are added up.

Fine aggregates : Fine aggregates shall be the fraction passing 2.36 mm sieve and retained on 75 micron sieve, consisting of crusher-run screeing, gravel, sand or a mixture of both.

Filler : Filler shall consist of finely divided mineral matter such as rock dust, hydrated lime or cement.

The filler shall be graded within the following limits :

IS Sieve	Per cent passing by weight
600 micron	100
300 micron	95-100
75 micron	85-100

The filler shall be free from organic impurities and have a Plasticity Index not greater than 4. The Plasticity Index requirement shall not apply if filler is cement or lime.

- When the coarse aggregate is gravel, 2 per cent by mass of total aggregate of portland cement or hydrated lime shall be added and the per centage of fine aggregate reduced accordingly.
- Cement or hydrated lime is not required when the gravel is limestone.

Aggregate gradation : The combined coarse and fine aggregates and filler shall produce a mixture to conform to the grading set forth in Table below :

Table Aggregate Gradation for Dense Bituminous Macadam

Sieve Designation	Per cent passing the sieve by weight
37.5 mm	100
26.5 mm	90-100
13.2 mm	56-80
4.75 mm	29-59
2.36 mm	19-45
300 micron	5-17
75 micron	1-7

Mix Design

Requirement of mix : Apart from conformity with grading and quality requirments of individual ingredients, the mix shall meet the requirements set out in Table below :

Table- Requirements of Dense Bituminous Macadam Mix

S.No.	Description	Requirements
1.	Marshall stability (ASTM Designation-D-1559) determined on Marshall specimens compacted by 75 compaction blows on each end	820 kg (1800 lb) minimum
2.	Marshall flow (mm)	2-4
3.	Per cent Air voids	3-5
4.	Minimum voids in mineral aggregates (VMA)	10 per cent - 12 per cent
5.	Per cent voids in mineral aggregates filled by Bitumen (VFB)	65-75
6.	Binder content per cent by weight of total mix	No less than 4.0 per cent

Binding content : The binder content shall be so fixed as to achieve the requirements of the mix set out in table above. Marshall method for arriving at the binder content shall be adopted, replacing the aggregates retained on 26.5 mm sieve by the aggregates passing 26.5 mm sieve and retained on 22.4 mm sieve.

Construction Operations

Preparation of base : The base on which Dense Bituminous Macadam is to be laid shall be prepared, shaped and conditioned to the specified lines, grades and cross sections.

- The surface shall be thoroughly swept clean free from dust and foreign matter using mechanical broom and dust removed or blown off by compressed air.

Tack coat : A tack coat over the base shall be applied.

Spreading : The mix transported from the hot mix plant to the site shall be spread by means of a self-propelled paver.

- The temperature of mix at the time of laying shall be in the range of 120°C-160°C.
- Mixes with a temperature of less than 120°C shall not be put into paver spreader.

Rolling : After spreading the mix by paver, it shall be thoroughly compacted by rolling with a set of rollers moving at a speed not more than 5 km/h. immediately following close to the paver.

- Rolling shall be continued till the density achieved is at least 98 per cent of that of laboratory Marshall specimen and all roller marks are eliminated.
- Skin patching of an area that has been rolled will not be permitted.
- Rolling operations shall be completed in all respects before the temperature of the mix falls below 100°C.

Opening to Traffic

Traffic may be allowed after completion of the final rolling when the mix has cooled down to the surrounding temperature.

- The Dense Bituminous Macadam shall be provided with an appropriate wearing course as early as possible prior to regular opening to normal traffic and/or impending rain.

Surface Dressing

Single and Two-Coat Surface Dressing using Bitumen

Description : This work shall consist of the application of one coat or two coats of surface dressing, each coat consisting of a layer of bituminous binder sprayed on base prepared previously, followed by a cover of stone shippings properly rolled to form a wearing course.

Materials

Binder : The binder shall be straight-run bitumen of a suitable grade appropriate to the region, traffic, rainfall and other environmental conditions.

Stone chippings : The stone chippings shall conform the requirement in bituminous macadam. The Stone Polishing Value shall not be less than 55.

Quantities of materials : The quantities of materials used for this work shall be as specified in Table below :

Table- Size Requirements of Stone Chippings for Surface Dressing Using Bitumen

S. No.	Type of construction	Normal Size of Stone chippings	Specifications
1.	Single coat surface dressing or the first coat of two-coat surface dressing	13.2 mm	100 per cent passing through 22.4 mm sieve & retained on 11.2 mm sieve
2.	Second coat of two-coat surface dressig (also used as a renewal coat).		100 per cent passing through 13.2 mm sieve & retained on 5.6 mm sieve

Table XV: Quantities of material required for 10 sqm. of road surface for surface dressing using bitumen.

S.No.	Type of construction	Binder	Stone] chippings
1.	Single coat surface dressing or the first coat of two-coat surface dressing	18.0 kg	0.15 cu.m
2.	Second coat of the two-coat surface dressing (also used as a renewal coat)	10.0 kg	0.10 cu.m.

Construction operations

Weather and seasional limitations : The base on which the surface dressing is to be laid shall be prepared, shaped and conditioned to the specified lines, grade and cross-section.

- Priming coat, where needed, shall be provided.
- Where the existing surface shows sings of fatting up, this shall be rectified.
- The bituminous primed surface to be dressed shall be thoroughly cleaned either by using a mechanical broom or any other approved equipment.
- Dust removed in the process shall be blown off with the help of compressed air.

Application of binder : Bitumen shall be heated to 150°C-163°C and sprayed on the dry surface in a uniform manner with the help of self-propelled machanical sprayers having self-heating arrangement and bitumen pressure pump and spraying bar with nozzles having constant volume or pressure system capable of spraying bitumen uniformly at the rates specified.

- Excessive deposites of binder caused by stopping or starting of the sprayer or through leakages or any other reason shall be corrected by blotting with sand before the stone chippings are spread.

Application of stone chippings : Immediately after the application of binder, stone chippings in a dry and clean state shall be spread uniformly on the surface by means of a self-propelled or towed mechanical grit spreader capable of spraying uniformly so as to cover the surface completely.

Rolling : Immediately after the application of the cover material, the entire surface shall be rolled with a 80-100 kN smooth wheeled steel roller or 80-100 kN static weight vibratory roller. Rolling shall commence at the edges and progress towards the centre except in superelevated and uni-directional cambered portions where it shall proceed from the lower edge to the higher edge. Each pass of the roller shall uniformly overlap not less than one-third of the track made in the preceding pass. While rolling is in progress, additional chippings shall be spread by hand in necessary quantities required to make up irregulatities.

- Rolling shall continue until all aggregate particles are firmly embedded in the binder and present a uniform closed surface.

Application of second coat of surface dressing : Where surface dressing in two coats is specified, the second coat shall be applied immediately after laying of the first coat. The construction operations for the second coat shall be the same as described for 1st coat.

Opening to traffic : Traffic shall not be permitted to run on any newly surface dressed area until the following day.

Two-Coat Surface Dressing Using Cationic Bitumen Emulsion.

This work shall consists of two-coat surface dressing, each coat consisting of a layer of cationic bitumen emulsion binder sprayed on a base prepared previously, followed by a cover of stone aggreagate, properly rolled to form a wearing course.

Material

Binder : Th binder shall be a Cationic type bitumen emulsion of Rapid Setting (RS) grade and having bitumen content 60 per cent minimum by weight.

Aggregate

Wet aggregates can be used for surface dressing with Cationic bitumen emulsions and when aggregates are dusty, they sould be cleaned by washing or by sprinkling water copiously. If the road surface is dry, a light sprinkling with water shall be done.

The aggregate shall conform to the size given in table below :

Table : Requirements of Stone Chippings for Surface Dressing Using Bitumen Emulsion

1. For first coat-13.2 mm size	Passing 22.4 mm sieve and retained on 11.2 mm sieve.
2. For second coat 6.7 mm size	Passing 9.5 mm sieve and retained on 2.36 mm sieve.

Quantities of Materials : The quantities of materials used for this work shall be as given in table below :

Table : Requirements of Materials Using Bitumen Emulsion

Material for 10 sq. m. area	For First Coat	For Second Coat
Cationic Bitumen Emulsion	12 to 14 kg	16 to 18 kg
Aggregates	0.10 to 0.12 cu.m.	0.06 to 0.08 cu.m.

Construction Operations

Weather limitations : Cationic bitumen emulsions should not normally be stored below 0°C.

- Surface dressing with Cationic bitumen emulsion should be carried out only when the atmospheric temperature is above 10°C.
- The work can be carried out when the base is damp.
- All standing water in depressions shall, however, be removed.

Preparation of base : The existing base on which surface dressing is to be laid shall be prepared, shaped and corrected to a uniform grade and camber.

The surface shall be cleaned to remove all loose particles, dust and foreign matter. It is preferable to spray water on the surface to settle the loose dust and also to expose clean surface of aggregates in the case of granular base courses.

Preparation of binder : Before opening, the Cationic bitumen emulsion drums should be rolled at slow speed, to and fro, for a distance of about 10 metres, 5 to 6 times to mix the contents properly.

<u>First Coat</u>

(A) **Application of Binder :** Cationic bitumen emulsion binder can be applied on wet road surface. If the road surface is dry, light sprinkling with water shall be done. Cationic bitumen emulsion shall be sprayed uniformly on the prepared base by mechanical sprayers.

(B) **Application of aggregate :** Immediately after spraying of Cationic emulsion, aggregate of 13.2 mm size shall be spread uniformly and evenly by mechanical means to cover the surface completely and evenly. Any oversize aggregate, if seen shall be removed.

(C) **Rolling :** Immediately after the application of cover material, the surface shall be rolled with 80-100 kN roller preferably by smooth wheeled tandem type.

- The rolling shall begin at the edge and proceed towards the centre, parallel to the centre line except in superlegvated and unidirectional cambered portions where it shall proceed from the lower edge to the higher edge. While rolling, aggregates shall be added or removed so as to ensure a uniformly covered surface.
- Each-pass of roller shall uniformly overlap not less than one third of the track made in the preceding pass.
- Rolling shall continue for just enough time to embed the aggregates in the binder and present a uniform closed surface.
- Excessive rolling, resulting in crushing of aggregates shall be avoided.

Second coat

(A) **Time interval :** The second coat of surface dressing shall be applied on the same day as the first coat but at least not earlier than one hour after finishing the rolling of the first coat.

(B) **Application of emulsion :** Traffic shall not be allowed on the first coat before the application of second coat. If the aggregates of the first coat appear to be loose and unbounded in few spots, the same shall not be disturbed.

- The Cationic bitumen emulsion for second coat shall be sprayed by mechanical sprayer.

(C) **Application of aggregate :** Immediately after the application of emulsion, 6.7 mm size aggregates shall be spread uniformly by mechanical means to cover the whole surface evenly.

(D) **Rolling :** Rolling shall start soon after spreading the aggregates and all operations carried to achieve a unifrom closed surface.

(E) **Finishing :** After one pass of the roller, depression shall be filled with 6.7 mm size aggregate. If excess of aggregate is found in isolated spots, the bigger size aggregates shall be removed to give a uniform surface. Finish rolling on the next day helps to give a firm surface.

Opening to traffic : The road may be opened to traffic 4 hours after completing rolling of the second coat, but preferably the next day after 24 hours.

Surface Dressing with Precoated Aggregate using Bitumen

The work shall consit of application of either a single coat or two coats of surface dressing over a previously ~~prepared base, each coat consisting of application~~ of bituminous binder sprayed on a base previously prepared followed by a cover of precoated materials properly rolled to form a wearing corse.

- The techinique is basically the same as conventional surface dressing except that a small quantity of binder is used for precoating the aggregates. However, it is quite different from premix construction.

Binder : The binder shall be straight run bitumen of a suitable grade appropriate to the region, traffic and other environmental conditions.

Precoating : The aggregates shall be precoated with 0.75 to 1.00 per cent of their weight with binder and shall not be precoated simultanously with the surface dressing operation.

- The precoated aggregates shall be allowed to cure for at least one week so that they become non-sticky to facilitate easy spreading like normal uncoated aggregates.
- The aggregates, free of dust or fine particles shall be preheated to 160°C for precoating and then mixed with bitumen binder (0.75 to 1.00 per cent by weight of aggregate) heated to its application temperature. The aggregates and binder shall be thoroughly mixed in a mixer of approved type till the aggregates are uniformly coated.

Construction operations

Application of binder : After deducting the binder quantity consumed in precoating aggregates for first coat from the quantities specified in table-XV, remaining binder of first coat shall be heated to 150°C- 165°C and sprayed on the clean and dry surface in a uniform manner preferably with the help of mechanical sprayers having self-heating arrangement, bitumen pressure pump and spray nozzle bar capable of spraying bitumen uniformly at the rates and temperatures specified.

Application of aggregate : Immediately after the application of binder, aggregates in a dry and clean state shall be spread uniformly on the surface preferably by means of a mechanical gritter capable of spreading uniformly at the rates specified so as to cover the surface competely.

Rolling : Rolling will be done an usual

Application of second coat of surface dressing : The second coat of surface dressing, where specified, shall be applied immediately after the first coat or soon after, depending upon the conditions at site and type of binder used. The construction operations for the second coat shall be the same as in 1st coat except that the rate of application of aggregates and binder for second coat shall be as specified in table-XV after deducting binder consumed for precoating the aggregates required for second coat.

Open-graded Premix Carpet using Bitumen

Scop : This wok shall consist of laying and compacting an open-graded carpet of 20 mm thickness in a single course composed of suitable small-sized aggregates premixed with a bituminous binder on a previously prepared base to serve as a wearing course.

Materials

Binder : The binder shall be bitumen of a suitable grade appropriate to the region, traffic, rainfall and other environmental conditions.

Aggregates : The coarse aggregate shall conform to the requirent of BM. (bitumnums macadam)

Proportioning of materials :

The materials shall be proportioned as per quantities given in Table below :

Table : Quantities of Materials Required for 10m^2 of Road Surface for 20 mm Thick Open-Graded Premix Carpet Using Bitumen

Aggregates for Carpet	
(a) Stone chipping 13.2 mm size; passing 22.4 mm sieve and retained on 11.2 mm sieve	0.18 m^3
(b) Stone chippings 11.2 mm size passing 13.2 mm sieve and retained on 5.6 mm sieve	0.09 m^3
Total	**0.2 m^3**
Binder for Premixing (quantities in terms of straight run bitumen)	
(a) For 0.18 m^3 of 13.2 mm size stone chippings at 52 kg per m^3	9.5 kg
(b) For 0.09 m^3 of 11.2 mm size stone chippings at 56 kg per m^3	5.1 kg
Total	**14.6 kg**

Construction operations

Preparation of base : The underlying base on which the bituminous carpet is to be laid shall be prepared, shaped and conditioned to the specified lines, grade and cross-section. A prime coat where needed shall be applied. A tack coat shall be applied over the base preparatory to laying of the carpet.

Preparation of premix : Hot mix plant of appropriate capacity and type shall be used for the preparation of mix material. The hot mix plant shall have separate dryer arrangement for heating aggregates and pugmil for mixing aggreates and binder.

- The temperature of binder at the time of mixing shall be in the range of 150°C to 163°C and that of the aggregates in the range of 155°C to 163C provided that the difference in temperature between the binder and aggregates at no time exceeds 14°C.
- Mixing shll be thorough to ensure that a homogeneous mixture is obtained in which all particles of the aggregates are coated uniformly and the discahrge temperature of mix shall be between 130°C and 160°C.

Spreading and rolling : The mixed material shall be spread by suitable means. As soon as sufficient length of bituminous material has been laid, rolling shall commence with 80-100 kN rollers, preferaly of smooth wheel tandem type,. Rolling shall begin at the edge and progress toward the centre longitudinally, except that on the superelevated and uni-directional cambered portions, it shall progress from the lower to upper edge parallel to the centre line of the pavement.

When the roller has passed over the whole area once, any high spots or depressions which become apparent shall be corrected by removing or adding premixed materisl.

- Rolling shall then be continued until the entire surface has been rolled to compaction and all the roller marks eliminated. In each pass of the roller, preceding track shall be overlapped uniformly by at least 1/3 width.
- The roller wheels shall be kept damp to prevent the premix from adhering to the wheels and being picked up. Rolling operations shall be completed in every respect before the temperature of the mix falls below 100°C.

  A seal coat shall be applied to the surface immediately after laying the carpet.

  No traffic shall be allowed on the road till the seal coat has been laid. After the seal coat is laid, the road shall be opened to traffic.

20 mm Thick Premix Carpet using Cationic Bitumen Emulsion

Scope : This work shall consist of laying and compacting an open graded premix carpet of 20 mm thickness with Cationic bitumen emulsion placed on a previously prepared base in accordance with the requirements of these Specifications to serve as wearing course.

Materials

Binder : The binder shall be Cationic bitumen emulsion of Medium Setting (MS) grade and having bitumen content 60 per cent minimum by weight. For liquid seal coat MS grade can be used, but it is preferable to use Rapid Setting (RS) grade of Cationic bitumen emulsion. However, for premix seal coat Slow Setting (SS) grade Cationic bitumen emulsion shall be used.

The water absorption (of coarse aggregate) value shall be limited to a maximum of 1 per cent. Water absorption up to 2 per cent may be permitted in exceptional cases only.

Fine aggregates for seal coat shall be crushed stone chips or coarse sand, clean , uncoated and free from clay, dust and other deleterious matter.

Quantities of materials required : The materials shal be proportioned as per quantities given in table below :

Table-X Quantities of Aggregates for 10 m² Area

(A) Premix Carpet	
(a) Coarse aggregate 13.2 mm size; passing IS 22.4 mm sieve and retained on IS 11.2 mm sieve	0.18 m^3
(b) Coarse aggregate 11.2 mm size; passing IS 13.2 mm sieve and retained on IS 5.6 mm sieve	0.09 m^3
(B) For Seal Coat :	
(a) Liquid seal coat :	
Crushed fine aggregates 6.7 mm size; passing IS 9.5 sieve and retained on IS 2.36 mm	0.06 m^3
(b) Premix seal coat :	
Coarse sand or stone grit passing 2.36 mm sieve and retained on 180 micron sieve	0.06 m^3

Table-Y Quantities of Binder

	For 10m² area
(A) For Tack Coat	
(i) Normal bituminous surfaces	2.0 to 2.5 kg
(ii) Dry and hungry bituminous surface	2.5 to 3.0 kg
(iii) Granular surfaces treated with primer	2.5 to 3.0 kg
(iv) Non bituminous surfaces :	
(a) Granular base (not primed)	3.5 to 4.0 kg
(b) Cement concrete surface	2.5 to 4.0 kg
(B) For Premix Carpet :	20 to 23 kg
(C) For Seal Coat :	
(a) for liquid seal coat	12 to 14 kg
(b) for premix seal coat	10 to 12 kg

Construction operations

Cationic bitumen emulsions shall not normally be stored below 0°C. Premix carpet work with Cationic emulsion shall be carried out only when the atmospheric temperature is above 10°C.

Preparation of base : The underlying base on which the premix carpet is to be laid shall be prepared, shaped and conditioned to the specified lines, grades and cross sections. The cleaned surface can be finally washed with water, if it is readily available.

Tack coat : Tack coat shall be applied not earlier than 10 minutes before spreading the premix. On water bound macadam surface, water shall be sprayed to make the surface damp before applying tack coat.

Preparation of premix : Premixing of Cationic bitumen emulsion and aggregates can be done in a suitable mixer such as cold mixing plant concrete mixer or by shovels . However, for large works, continuous mixing operation can be done either in batch or continuous mixer units suitable for emulsion mixes.

When using concrete mixer for preparing the premix, 0.135 cu.m. (0.09 cu.m. of 13.2 mm size and 0.045 cu.m. of 11.2 mm size) of aggregates per batch may be used as this quantity will cover 5 sq.m. of road surface with 20 mm average thickness.

Spreading of mix : The premixed cationic bitumen emulsion and aggregates shall be spread within 10 minutes of applying the tack coat. The mix is easily workable for about 20 minutes after mixing and hence all levelling, raking, etc. should be completed within this time for easy workability.

The mix should be spread uniformly to the desired thickness, grades and crossfall (camber) making due allowance for extra quantity required to fill up depressions, if any.

Rolling : The rolling shall start immediately after laying the premix. Smooth wheeled tandem roller of 80-100 kN shall be used preferably, though three wheeled steel roller of equivalent capacity can also be used. While rolling, wheels or roller should be clean and kept moist to prevent the premix from adhering to the wheels and being picked up.

Rolling shall commence at the edges and progress towards the centre longitudinally except in case of superelevated and unidirectional cambered sections where rolling shall be done from lower edge towards the higher edge parallel to the centre line of the road.

After one pass of roller over the whole area, depressions or uncovered spots should bve corrected by adding premix material. Rolling shall be continued until the entire surface has been rolled to compaction and all the roller marks eliminated. In each pass of the roller, preceding track shall be overlapped uniformly by at least 13 width.

Seal coat : A seal coat, liquid or premix type, shall be applied 4 to 6 hours after laying the premix carpet.

Liquid seal coat : Quantities of the materials required are given in Table-X and Y.

- Immediately after spraying emulsion, stone chips in a clean state shall be spread uniformly, preferably by means of a mechanical gritter, otherwise manually so as to cover the surface completely. Start rolling with 60-80 kN roller soon after spreading the chips.

Premix seal coat : The quantities of aggregate and binder to be used shall be as given in Table-X and Y. Grit or sand used for premix seal coat should be made thoroughly wet with water before mixing with emulsion of slow setting grade.

Rolling shall be continued till the premix material completely seals the voids in the premix carpet and smooth uniform surface is obtained.

Traffic should not be allowed over the premix surface with or without seal coat, for 6 to 8 hours after rolling.

Semi-Dense Bituminous Concrete

This work shall consist of construction in a single course of semidense bituminous concrete binder/ wearing course on a previously prepared bituminous.

Materials

Coarse aggregates : Coarse aggregate shall be as per BM requirement. The maximum value of water absorption in coars aggregate shall be 1 per cent. However, water absorption upto a maximum of 2 per cent may be permitted in exceptional cases only.

Aggregates gradation : The mineral aggregates including filler shall be so graded or combined as to conform to the grading set forth in table Z.

Table-Z Aggregates Gradation for Semi-Dense Bituminous Concrete

IS Sieve Designation	Per cent by weight passing the sieve		
	Grading-1	Grading-2	Grading-3
22.4 mm	–	100	100
13.2 mm	100	85-100	79-100
11.2 mm	88-100	70-92	68-90
5.6 mm42-64	42-64	33-55	
2.8 mm22-38	22-38	22-38	
710 micron	11-24	11-24	6-22
355 micron	7-18	7-18	4-14
180 micron	5-13	5-13	2-9
90 micron	3-9	3-9	0-5

Note : *Grading 1 shall be adopted for 25 mm compacted thickness and Grading 23 for higher thickness.*

Mix Design

Requirements of mix : Semi-dense bituminous concrete mix shall be properly designed so as to satisfy the criteria laid down in Table ∝

Table-∝ Requirements of Semi-Dense Bituminous Concrete Mix

1. Marshall Stability determined on Marshall specimens compacted with 75 compaction blows on each end	820 kg Minimum
2. Marshall flow (mm)	2-4
3. Per cent air voids in mix	3-5
4. Per cent air voids in mineral aggregate (VMA) (Minimum)	13-15 (for 13.2 mm max size) 11-13 (for 22.4 mm max size
5. Per centage voids in mineral aggregates filled with bitumen (VFB)	65-75
6. Binder content, per cent by weight of mix	Not less than 4.0 per cent

Construction Operations

Preparation of base : The base on which semi-dense bituminous concrete is to be laid shall be prepared, shaped and conditioned to the specified lines, grades and cross-sections.

A tack coat as shall be applied on the base

Preparation of mix, spreading, rolling etc. will be as per DBM (Dense Bituminious Macadon)

Bituminous Concrete

This work shall consist of constructing in a single layer, bituminous concrete (asphaltic concrete) of thickness 25-100 mm on previously prepared bituminous course.

Materials

Bitumen : Will be same as for DBM

Coarse aggregates : Shall be same as per DBM. The aggregates shall satisfy the physical requirements as given in Table for DBM except that the maximum value for the water absorption should be 1 per cent.

Fine aggregates : Shall be same as per DBM

Filler : Shall be same as per DBM

Aggregates gradation : The mineral aggregates, including mineral filler shall be so graded or combined as to conform to the grading set forth in table below :

Table: Aggregate Gradation for Bituminous Concret

Sieve Designation	Per cent] passing tbe sieve by weight
26.5 mm	100
19 mm	90-100
9.5 mm	56-80
4.75 mm	35-65
2.36 mm	23-49
300 micron	5-19
75 micron	2-8

Mix Design

Requirement of mix : Apart from conformity with the grading and quality requirements of individual ingredients, the mix shall meet the requirements set forth in Table below :

Table : Requirements of Bituminous Concrete Mix

S.No.	Description	Requirement
1.	Marshal stability determined on Marshall specimens compacted by 75 compaction blows on each end	820 kg (Minimum)
2.	Marshall flow (mm)	2-4
3.	Per cent air voids in mix	3-5
4.	Per cent voids in mineral aggregate (VMA)	Minimum 11-13 %
5.	Per cent voids in mineral aggregates filled by bitumen (VFB)	65-75
6.	Binder content, per cent by weight of total mix	Minimum 4.5
7.	Water Sensitivity, Loss of stability on immersion in water at 60°C	Min. 75 per cent retained strength
8.	Swell Test	1.5 per cent Max.

Binder content : The binder content shall be so fixed as to achieve the requirements of the mix set forth in Table above. Marshall method for arriving at the binder content shall be adopted.

Construction Operations

Preparation of base : The base on which bituminous concrete is to be laid shall be prepared, shaped and conditioned to the specified levels, grade and crossfall (camber).

The surface shall be thoroughly swept clean free from dust and foreign matter usng mechanical broom and dust removed by mechanical means or blown off by compressed air.

Tack coat : A tack coat shall be applied over the base

Preparation of mix, spreading, Rolling will be as per DBM

Traffic may be allowed immediately after completion of the final rolling when the mix has cooled down to the surrounding temperature.

Seal Coat

This work shall consist of application of a seal coat for sealing the voids in a bituminous surface laid to the specified levels, grade and cross fall (camber)

Seal coat shall be of either of the two types as specified below :

(A) Liquid seal coat comprising of an application of a layer of bituminous binder followed by a cover of stone chippings.

(B) Premixed seal coat comprising of a thin application of fine aggregates premixed with bituminous binder.

- The quantity of bitumen shall be 9.8 kg and 6.8 kg per 10 square metres area for Type (A) and Type (B) seal coat respectively.

Stone chipping for type (A) seal coat : The stone chippings shall consist of angular fragments of clean, hard, tough and durable rock of uniform quality throughout. They should be free of elongated or flaky pieces, soft or disintegrated stone, organic or other deleterious matter. Stone chippings shall be of 6.7 mm size defined as 100 per cent passing through 11.2 mm sieve and retained on 2.36 mm sieve. The quantity used for spreading shall be 0.09 cubic metre per 10 square metres area. The chippings shall satisfy the quality requirements spelt out in Tablefor BM except that the upper limit for water absorption value shall be 1 per cent.

Aggregate for type (B) seal coat : The aggregates shall be sand or grit and shall consist of clean, hard, durable, uncoated dry particles and shall be free from dust, soft or flaky/elongated material, organic matter or other deleterious substances. The aggregate shall pass 2.36 mm sieve and be retained on 180 micron sieve, The quantity used for premixing shall be 0.06 cubic metre per 10 square metre area.

Construction Operations

Preparation of base : The seal coat shall be applied immediately after the laying of bituminous course which is required to be sealed. Before application of seal coat materials, the surface shall be cleaned free of any dust or other extraneous matter.

Construction of type (A) seal coat : The binder shall be heated in boilers of suitable design, to the temperature appropriate to the grade of bitumen and the seal coat applied in.

Construction of type (B) seal coat : Mixer of appropriate capacity and type shall be used for preparation of mix material. The plant shall have separate dryer arrangement for heating aggregate and pugmill for mixing aggregate and binder.

- The binder shall be heated in boilers of suitable design, to the temperature appropriate to the grade of bitumen. Also the aggregates shall be dry and suitably heated to a temperature before the same are placed in the mixer. Mixing of binder with aggregates to the specified proportions shall be continued till the latter are thoroughly coated with the former.
- The mix shall be immediately transported from the mixing plant to the point of use and spread uniformly on the bituminous surface to be sealed.
- As soon as sufficient length has been covered with the premixed material, the surface shall be rolled with 80-100 kn smooth-wheeled roller. Rolling shall be continued till the premixed material completely seals the voids in the bituminous course and a smooth uniform surface is obtained.
- In the case of type (B) seal coat, traffic may be allowed soon after final rolling when the premixed material has cooled down to the surrounding temperature. However, as regards type (A) seal coat, opening of traffic will be done the next day.

Bitumen Mastic

- This work shall consist of constructing a single layer of 25 mm to 50 mm thick Bitument Mastic wearing course for road pavements and bridge decks.
- The bitumen mastic shall be laid over a Dense Bituminous Macadam base in cse of road pavements and over a Cement Concrete base in bridge decks.
- The Bitumen Mastic is an intimate homogeneous mixture of selected well-graded aggregates and bitumen in such proportions as to yield a plastic and voidless mass, which when applied hot can be trowelled and floated to form a very dense impermeable surfacing

Materials

Bitumen : The binder shall be straight-run bitument of a suitable grade or industrial bitumen. It shall be paving bitumen of Grade S 35 (30/40 penetration grade). The requirements of physical properties of bitumen are as given in Table below.

Table : Requirements of Physical Properties of Bitumen

S.No.	Characteristic	Requirement
1.	Penetration at 25°C (in 1/100 cm)	20 to 40
2.	Softening point (Ring and Ball Method)	50°C to 90°C
3.	Ductility at 27°C (Minimum)	10 cm
4.	Loss on heating (Maximum)	3 per cent
5.	Solubility in CS_2 (Minimum)	99 per cent

Coarse aggregates : The coarse aggregates shall consists of hard, durable crushed stones, which shall be clean, strong, free of disintegrated pieces, organic and other deleterious matter and adherent coatings.

- They shall be hydrophobic and of low porosity.
- The per centage and grading of the coarse aggregates to be incorporated in the bitumen mastic depending upon the thickness of the finished course is given in table below.

Table : Grading and Per centage of Coarse Aggregates

S.No.	*Type of work*	*Thickness of Finished Course (mm)*	*Per centage of Coarse Aggregates*	*Grading of Coarse Aggregates*	
				Passing Is sieve	*Percentage*
1.	Wearing course	(a) 25-40	(a) 30-40	19 mm	100
	for pavement			13.2 mm	88-96
	& bridge deck	(b) 41-50	(b) 40-50	2.36	0-5

Fine aggregates : The fine aggregates shall be the fraction pasing 2.36 mm and retained on 75 micron sieve consisting of crusher run screening, natural sand or a mixture of both.

Filler : The filler shall be limestone power passing 75 micron sieve and shall have a calcium carbonate content of not les than 80 per cent by weight.

The grading of the fine aggregates inclusive of filler shall be as given in Table below :

Table : Grading of Fine Aggregates (Inclusive of Filler)

S.No.	I.S. Sieve Passing	I.S. Seive Retained	Per centage by Weight
1.	2.36 mm	60 micron	0-25
2.	600 micron	212 micron	5-35
3.	212 micron	75 micron	10-20
4.	75 micron	--	30-50
	Bitumen Binder		14-17

Mix Design

Hardness number : The bitumen mastic shall have a hardness number of 60 to 80 at 25°C without coarse aggregates and 10 to 20 at 25°C after adding coarse aggregates.

Binder content : The binder content shall be so fixed as to achieve the requirements of the mix of 14 to 17 per cent by weight of total mix.

Construction Operations

Preparation of the base : The base on which bitumen mastic is to be laid shall be prepared, shaped and conditioned to the specified levels, grade and crossfall. In case of a cement concrete base, the surface shall be thoroughly swept and scraped clean and free of dust and other deleterious matter.

- Under no circumstances shall bitumen mastic be spread on a base containing a binder which will soften under high application temperatures.

Tack coat : A tack coat shall be applied on the base

Preparation of bitumen mastic : Preparation of bitumen mastic consists of two stages. The first stage shll be mixing of filler and fine aggregates and then heating the mixture to a temperature of 170°C to 200°C. Required quantity of bitumen shall be heated to 170°C to 180°C and added to the heated aggregates.

- They shall be mixed and cooked in an approved type of mechanically agitated mastic cooker for some time till the materials are thoroughly mixed.
- Initially the filler alone is to be heated in the cooker for an hour and then half the quantity of binder is added.
- After heating and mixing for some time, the fine aggregates and the balance of binder are to be added and further cooked for about one hour.
- The second stage is incorporation of coarse aggregates and cooking the mixture for a total period of 3 hours. During cooking and mixing, care shall be taken to ensure that the contents in the cooker are at no time heated to a temperature exceeding 200°C.
- In case where the material is not required for immediate use, it shall be cast into blocks with filler, fine aggregates and binder, weighing about 25 kg each. These blocks when required to be used, have to be reheated in the mechanically agitated cooker to a temperature of not less than 170°C and not more than 200°C, thoroughly incorporated with the requisite quantity of coarse aggregates and mixed continuously for not less than an hour.
- Mixing shall be continued until laying operations are completed so as to maintain the coarse aggregates in suspension. At no stage during the process of mixing shall the temperature exceed 200°C.
- The bitumen mastic blocks (without coarse aggregates) shall show on analysis a compositin within the limits as given in Table below :

Table : Composition of Bitumen Mastic Blocks Without Coarse Aggregates

I.S.Sieve Passing	I.S. Sieve Retained	Per centage by weight	
		Minimum	Maximum
2.36 mm	600 micron	0	22
600 micron	212 micron	4	30
212 micron	75 micron	8	18
75 micron	–	25	45
Bitumen Content		14	17

The mix shall be transported to the laying site in a towed mixer transporter having arrangement for stirring and keeping the mix hot during transportation.

Spreading : The bitumen mastic shall be deposited directly on the prepared base immdiately in front of the spreader where it is spread uniformly by means of wooden floats to the required thickness.

- The mix shall be laid in one metre width contained between angle iron of sizes sitable to retain the required thickness.
- The temperature of the mix at the time of laying shall be around 170°C. In case blowing takes place while laying the bitumen mastic, the bubbles shall be punctured while it is hot and the surface treated well.

Joints : It shall be ensured that all construction joints are properly and truly made.

- These joints shall be made by warming existing bitumen mastic by the application of excess quantity of the bitumen mastic mix which afterwards shall be trimmed off to make it flush with the surfaces on either side.

Surface finish : The bitumen mastic surface can be slippery after floating ; in order to provide resistance to skidding, the bitumen mastic after spreading, while still hot, and in plastic condition shall be covered with a layer of stone aggregates of 9.5 mm size (passing 13.2 mm sieve and retained on 6.7 mm sieve).

- Hard stone ships of approved quality precoated with bitumen at the rate of two per cent of S-65 to S-90 penetration grades at 25°C at the rate of 0.005 cu.m. per 10 sq.m. shall be rolled or otherwise pressed into the surface of the mastic layer when the temperature of bitumen mastic is between 80°C and 100°C

  The traffic may be allowed after completion of the work when the bitumen mastic has cooled down to the surrounding temperature.

Slurry Seal

This work shall consist of mixing emulsified bitumen, well-graded fine aggregates (with mineral filler) and water, spreading the mixture and rolling on a pavement surfacing as a surface treatment.

The materials for slurry seal immediately prior to mixing shall conform to the following requirement.

Emulsified bitumen : The emulsified bitumen shall be a Cationic rapid setting type. It shall be capable of producing a slurry as would develop early resistance to traffic and rain and is sufficiently stable to permit mixing with the specified aggregates, without breaking during the mixing and laying processes.

Water shall be of such quality that the bitumen will not separate from the emulsion before the slurry seal is in place of work.

Aggregates : The aggregates shall be crushed igneous rock, limestone or slag and may be blended, if required, with clean, sharp, naturally occuring sand free from silt, clay or other fine material to produce a grading as given in Table below. It shall meet the requirements of the film stripping test and suitable amount and type of anti-striping agent added, as may be needed.

Table : Aggregates Grading for Slurry Seal

Sieve size	Per centage by mass passing Finish thickness of sealing	Per centage by mass passing Finish thickness of sealing
	3mm	1.5 mm
4.75 mm	100	–
3.35 mm	80-100	100
2.36 mm	75-100	95-100
1.18 mm	55-90	70-95
600 micron	35-70	50-75
300 micron	20-45	30-35
150 micron	10-25	10-30
75 micron	5-15	5-15

Proportioning

The mixed material shall consist of aggregates, water and emulsified bitumen (about 180-250 litres per tonne of dry aggregates) and where necessary, Portland cement as an additive. The proportion of the additive (Portland cement) shall not normally exceed 2 per cent by weight of dry aggregates. The precise proportion of each constituent should be selected by laboratory tests and trials.

The various constituents shall be weighed in the needed proportions and mixed in a mechanical mixer so that all particles of the aggregates are uniformly coated and the slurry is of such consistency as can be satisfactorily laid. It shall be a semi-fluid, homogeneous mass with no emulson run-off.

- The pavement surface shall be suitably prepared by removing any loose material, dust and vegetation after patching up any potholes, depressions etc. All manhole covers, inspection covers and gully gratings should be masked.

Laying

If required, a tack coat shall be applied. The rate of spread of tack coat shall depend on the surface to be treated. For bituminous surfaces, at the rate of 0.15 to 0.30 litres/sq.m. and for concrete surfaces at the rate of 0.4 to 0.6 litres/sq. m.

- Spreading shall be carried out by mechanical means. Spreading shall not be undertaken when the ground temperature falls below 4°C or where the surface contains standing water. The slurry shall be spread at a rate such that the cover of aggregates (dry mass equaivalent) is 4-6 kg/sq.m. for 3 mm thick work and 2-4 kg/sq.m. for 1.5 mm thick work.
- The slurry shall be rooled by a roller having an individual wheel load between 7.5 and 15.0 kN making at least six passes. Rolling shall commence as soon as the slurry has set sufficiently to ensure that rutting or excessive movement will not occur.

- Adequate steps shall be taken to avoid damage by traffic until such time that the mixture has cured sufficiently so that the slurry seal will not adhere to and be picked up by the tyres of vehicles.

CONCRETE PAVEMENT

Cement Concrete Pavement

The work shall consist of construction of unreinforced, dewel jointed, plain cement concret pavement in conformity with the lines, grades and cross sections.

Materials

Cement : Any of the following types of cement capable of achieving the design strength may be used , but the preference should be to use at least the 43 Grade or higher.

(i) Ordinary Portland Cement, 33 Grade, IS : 269.

(ii) Ordinary Portland Cement, 43 Grade, IS : 8112.

(iii). Ordinary Portland Cement, 53 Grade, IS : 12269.

- If the soil around has soluble salts like sulphates in excess of 0.5 per cent, the cement used shall be sulphate resistant.

Admixtures : Admixtures shall be permitted to improve workability of the concrete or extension of setting time on satisfactory evidence that they will not have any adverse effect on the proporties of concrete with respect to strength, volume change, durability and have no deleterious effect on steel bars.

Aggregates

- Aggregates for pavement concrete shall be natural material but with a Los Angeles Abrasion Test result not more than 35 per cent.
- The aggregates shall be free from chert, flint, chalcedony or other silica in a form that can react with the alkalies in the cement.
- In addition the total chlorides content expressed as chloride ion content shall not exceed 0.06 per cent by weight and the total sulphate content expressed as sulphuric anhydride (SO_3) shall not exceed 0.25 per cent by weight.

Coarse aggregate : Coarse aggregate shall consist of clean, hard, strong, dense, non-porous and durable pieces of crushed stone or crushed gravel and shall be devoid of pieces of disintegrated stone, soft, flaky, elongated, very angular or splintery pieces.

- The maximum size of coarse aggregate shall not exceed 25 mm for pavement concrete.
- No aggregate which has water absorption more than 2 per cent shall be used in the concrete mix.
- The aggregates shall be tested for soundness. After 5 cycles of testing the loss shall not be more than 12 per cent if sodium sulphate solution is used or 10 per cent if magnesium sulphate solution is used.

Fine aggregate : The fine aggregate shall consist of clean natural sand or crushed stone sand or a combination of the two. Fine aggregate shall be free from soft particles, clay, shale, loam, cemented particles, mica and organic and other foreign matter. The fine aggregate shall not contain deleterious substances more than the following :

Clay lumps	4.0 per cent
Coal and lignite	1.0 per cent
Material passing IS Sieve No. 75 micron	4.0 per cent

Water : Water used for mixing and curing of concrete shall be clean and free from injurious amount of oil, salt, acid, vegetable matter or other substances harmful to the finished concrete.

Joint sealing compound : The joint sealing compound, shall be of hot poured, elastomeric type or cold polysulphide type having flexibility, resistance to age hardening and durability.

Cement content : The cement content shall not be less than 350 kg per cu.m. of concrete.

Sub-base

The cement concrete pavement shall be laid over the sub-base. If the sub-base is found damaged at some places or it has cracks wider than 10 mm, it shall be repaired with fine cement concrete or bituminous concrete before laying separation layer. Prior to laying of concrete it shall be ensured that the separation membrane is placed in position and the same is clean of dirt or other extraneous materials and free from any damage.

Separation Membrane

A separation membrane shall be used between the concrete slab and the subbase. Separation membrane shall be impermeable plastic sheeting 125 microns thick laid flat without creases. Before placing the separation membrane, the sub-base shall be swept clean of all the extraneous materials using air compressor.

- Where overlap of plastic sheets is necessary, the same shall be at least 300 mm and any damaged sheeting shall be replaced.

Joints

Joints shall be constructed depending upon their functional requirement. The location of the joints should be transferred accurately at the site and mechanical saw cutting of joints done as per stipulated dimensions.

- It should be ensured that the full required depth of cut is made from edge to edge of the pavement.
- Transverse and longitudinal joints in the pavement and sub-base shall be staggered so that they are not coincident vertically and are at least 1m and 0.3 m apart respectively.
- Sawing of joints shall be carried out with diamond studded blades soon after the concrete has hardened to take the load of the sawing machine and personnel without damaging the texture of the pavement. Sawing operation could start as early as 6-8 hours depending upon the season.

Transverse joints

Transverse joints shall be contraction and expansion joints.

Contraction joints : Contraction joints shall consist of a mechanical sawn joint groove, 3 to 5 mm wide and 1/4 to 1/3 depth of the slab ± 5mm.

The contraction joints shall be cut as soon as the concrete has undergone initial hardening and is hard enough to take the load of joint sawing machine without causing damage to the slab.

Expansion joints : The expansion joints shall consist of a joint filler board and dowel bars. The filler board shall be positioned vertically with the prefebaricated joint assembiles along the line of the joint within the tolerances. The adjacent slabs shall be completely separated from each other by providing joint filler board. Space around the dowel bars, between the sub-base and the filler board shall be packed with a suitable compressible material to block the flow of cement slurry.

Transverse construction joint : Transverse construction joints shall be placed whenever concreting is completed after a day's work or is suspended for more than 30 minutes. These joints shall be provided at the regular location of contraction joints using dowel bars. The joint shall be made butt type. At all construction joints, steel bulk heads shall be used to retain the concrete while the surface is finished. The surface of the concrete laid subsequently shall conform to the grade and cross sections of the previously laid pavement. When positioning of bulk head/ stop-end is not possible, concreting to an additional 1 or 2 m length may be carried out to enable the movement of joint cutting machine so that joint grooves may be formed and the extra 1 or 2 m length is cut out and removed subsequently after concrete has hardened.

Longitudinal joint

The longitudinal joints shall be saw cut . The groove may be cut after the final set of the concrete. Joints chould be sawn to at least 1/3 the depth of the slab ± 5 mm.

Tie bars shall be provided at the longitudinal joints.

Dowel bars

Do well have small bne mild steel or round

- They shall be straight, free of irregulatities and burring, restricting, slippage in the concrete. The sliding ends shall be sawn or cropped cleanly with no protrusions outside the normal diameter of the bar. The dowel bar shall be supported on cradles/ dowel chairs in pre-fabricated joint assemblies positioned prior to the construction of the slabs or mechanically inserted with vibration into the plastic concrete by a method which ensures correct placement of the bars besides full re-compaction of the concrete around the dowel bars. Dowel bars shall be postioned at mid depth of the slab within a tolerance of ± 20 mm, and centered equally about intended lines of the joint within a tolerance of ± 25 mm. They shall be aligned parallel to the finished surface of the slab and to the centre line of the carriageway.

  Dowel bars, supported on cradles in assemblies, when subject to a load of 110 N applied at either end and in either the vertical or horizontal direction (upwards and downwards and both directions horizontally) shall conform to be within the following limits :

(i) Two-thirds of the number of bars of any assembly tested shall not deflect more than 2 mm per 300 mm length of bar

(ii) The remainder of the bars in that assembly shall not deflect more than 3 mm per 300 mm length of bar.

Dowel bars ahll be covered by a thin plastic sheath for at least two-thirds of the length from one end for dowel bars in contraction joints or half the length plus 50 mm for expansion joints. The sheath shall be tough, durable and of an average thickness not greater than 1.25 mm.

For expansion joints, a closely fitting cap 100 mm long consisting of waterproofed cardboard or an approved synthetic material like PVC or GI pipe shall be placed over the sheathed end of each dowel bar. An expansion space at least equal in length to the thickness of the joint filler board shall be formed between the end of the cap and the end of the dowel bar by using compressible sponge. To block the entry of cement slurry between dowel and cap it may be taped.

Tie bars

Tie bars in longitudinal joints shall be deformed steel bars of strength 415 MPa.

Tie bars projecting across the longitudinal joint shall be protected from corrosion for 75 mm on each side of the joint by a protective coating of bituminous paint.

Tie bars in longitudinal joints shall be made up into rigid assemblies with adequate supports and fixings to remin firmly in position during the construction of the slab. Alternatively, tie bars at longitudinal joints may be mechanically or manualy inserted into the plastic concrete from above by vibration using a method which ensures correct placement of the bars and recompaction of the concrete around the tie bars. Tie bars shall be postioned to remain within the middle third of the slab depth.

CONSTRUCTION EQUIPMENT

Excavation Equipment

The excavation equipment commonly used in highway projects include Bull dozer, Power shovel, Dragline, Clam shell, Hoe

Bull dozer

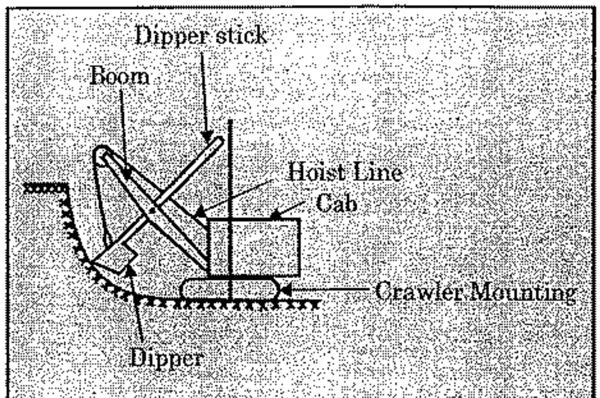
- Bull dozer can be used for shallow excavation work and for hauling the earth for relatively short distances.
- It is a versatile machine and can be used for many projects like for clearing site, opening up pilot roads, moving earth for short haul distances of about 100 m.

Scraper

- Scraper can also be used for shallow excavation work and for hauling the earth for relatively short distances.
- It is considered as one of the useful earth-moving equipment as it is self operating (can dig, haul and discharge the material in uniformly thick layers). However, they are not capable of digging very stiff material.

Power shovel

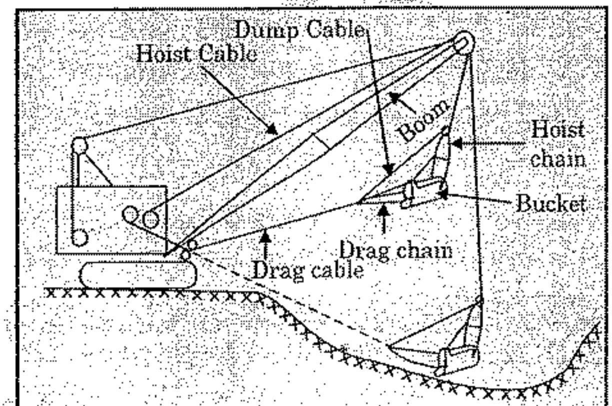
- Power shovel is used primarily to excavate earth of all classes except rock and to load it into wagons.
- It may be mounted on crawler tracks in order to move at low speeds.
- The power shovel can effectively operate to excavate earth from a lower level where it stands and when the depth of the face to be excavated is not too shallow.



Power Shovel

Dragline

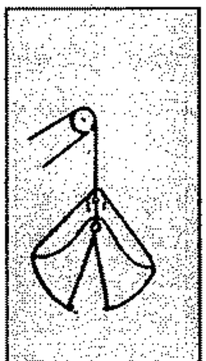
- Dragline is used to excavate soft earth and to deposit in nearby banks or to load into wagons.
- It may also be mounted on crawler.
- It can operate on natural ground while excavating from a pit with the bucket
- It is not necessary for the dragline to go into the pit in order to excavate.



Dragline

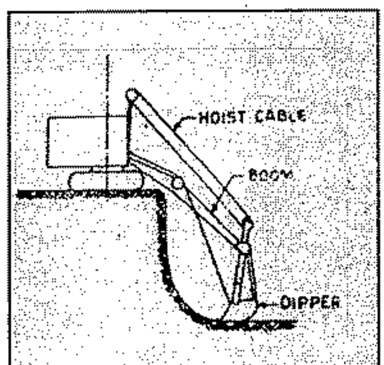
Clam shell

- Clam shell consists of a bucket of two halves or shell which are hinged together at top.
- This equipment is useful for excavation of soft to medium materials and loose material at or below existing ground surface.


ClamShell

Hoe

- Hoe is an excavating equipment of the power-shovel family.
- It is meant to excavate below the natural surface (machine is usually stationed at natural surface).
- It is capable of having precise control of depth of excavation at close range work.



Hoe

Note: Hoe can exert high tooth pressures and hence can excavate stiff material which normally can not be excavated by dragline.

Compacting Equipment

As soil compaction is achieved in the field by rolling, ramming or vibration, the compacting equipment may be classified as *rollers, rammers and vibrators*. Compaction of sands is achieved by watering, ponding and jetting.

Rollers: The principle of rollers is the application of pressure, which is slowly increased and then decreased. The various type of rollers which are used for compaction are (i) smooth wheeled rollers, (ii) pneumatic tyred rollers and (iii) sheepsfoot rollers.

(i) Smooth, Wheeled Rollers

- Smooth wheeled rollers are suitable to roll a wide range of soils, preferably granular soils and pavement materials for the various layers.
- Particularly found to be useful in compacting soils and other materials where a crushing action is advantageous.

(ii) Pneumatic Tyred Roller

- They are most suitable to compact non-plastic silts and fine sands.
- In addition to the direct pressure due to rolling, there is also a slight kneading action.

(iii) Sheepsfoot Roller

- Suitable to compact clayey soil.
- Efficiency of this type depends on its weight and no. of feet in contact with ground at a time.
- Thickness of compacting layer is kept about 5 cm more than the length of each foot.
- 24 or more no. of passes of the roller may be necessary to obtain adequate compaction.

Rammers : They are useful to compact relatively small areas and where the rollers cannot operate (eg., compaction of trenches, foundation and slopes).

Vibrators: They are most suited for compacting dry cohesionless granular material. Vibrator mounted roller gives the combined effects of rolling and vibration.

Watering : Watering (ponding or jetting) is considered to be an efficient method of compacting cohesionless sands.

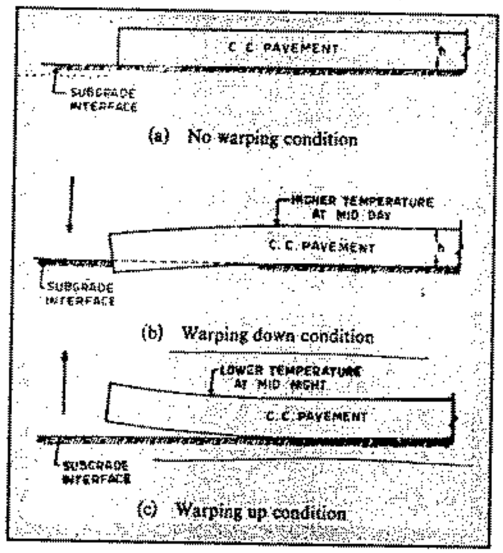
CONSTRUCTION OF JOINTS IN CEMENT CONCRETE PAVEMENTS

- The main provision of joints in cement concrete pavements is for expansion, contraction and warping of the slabs due to the variation in the temperature of slabs and moisture changes.
- Changes in atmospheric temp, (*i.e.*, either rise or fall) is a cyclic phenomena due to which pavement slabs tend to expand and contract horizontally as well as vertical.
- Joints are also provided in order to facilitate a break in the construction at the end of day's work or for any unexpected interruption to work progress.

Warping Stresses in Slab

- During the mid-day, the top of the pavement slab has higher temp. than the bottom of the slab. This causes top fibres of the slab to expand more than bottom fibres, and slab curls at edges.

  This phenomenon is known as warping down of the slab.
- In the mid night, temp. of the bottom of the slab is higher than the temp. of the top of the slab. This phenomenon is known as warping up of the slab.
- The weight of the pavement slab prevents the slab to take a warped shape and, hence, develops stresses in the slab which are known as ***warping stresses***.



(a) No warping condition

(b) Warping down condition

(c) Warping up condition

- When the slab is warped down, warping stresses are tensile in nature at the bottom of the slab.
- When the slab is warped up, the tensile stresses are developed at the top of the slab.

Note: The magnitude of warping stresses are maximum at the interior region and are minimum at the corner region.

Transverse Joints

Transverse joints are provided across the full width of pavement to minimise the temperature stresses in the pavement slab, expansion, contraction and warping.

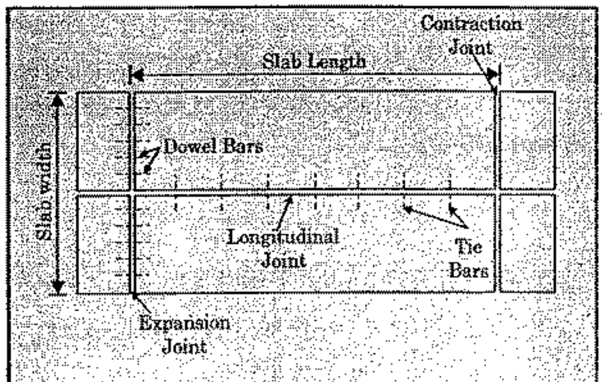
Transverse joints are further classified as:

(a) Expansion joint

(b) Contraction joint

(c) Warping joint

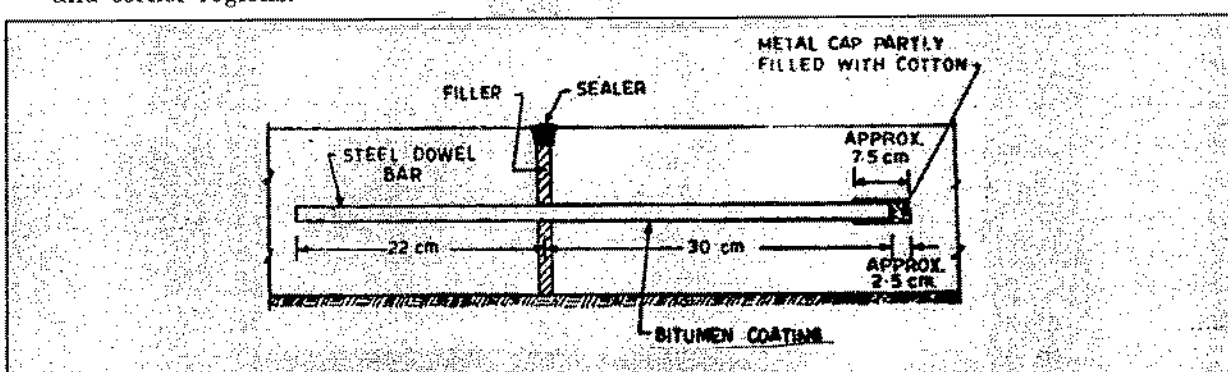
(d) Construction joint



Location of joints

Expansion Joints

- Expansion joints in our country is mainly provided at interval of 50 to 60 metre for smooth interface laid in winter and 90 to 120 metre for smooth interface laid in summer.
- For rough interface the spacing between expansion joints may be 140 m.
- The approximate gap width for this type of joints is from 20 to 25 mm.
- The stresses include due to the wheel loads at such joints are of very high order at the edge and corner regions.



Expansion joint with Dowel Bar

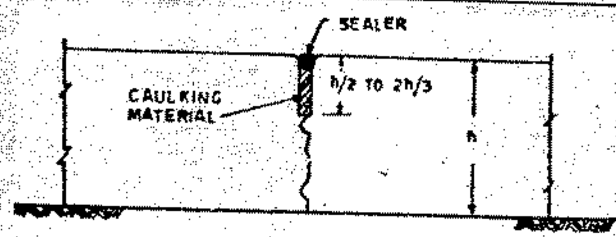
- The load transference across the transverse joint is carried out through a system of reinforcement provided at suitable intervals projecting in the concrete in longitudinal direction upto 60 cm length. Such a device is named as dowel bar.
- In the design, 40 percent of wheel load is expected to be taken up by the group of dowel bars and transferred to the adjoining slab.
- Spacing between the dowel bars is generally adopted as 30 cm.
- The size of the dowel varies with pavement thickness and it ranges between 20 to 30 mm.
- The total length of dowel bar varies between 40 cm to 73 cm depending upon the dowel diameter.

Contraction Joints

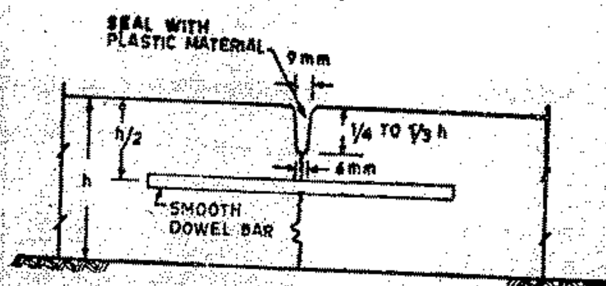
- These joints are spaced closer than expansion joints.
- Load transference is provided through the physical interlocking by the aggregates projecting out at the joints faces.
- As per IRC specifications, the maximum spacing of contraction joints in unreinforced Cement Concrete slabs is 4.5 m and in reinforced slab of thickness 20 cm is 14 m.
- It is recommended to provide contraction joints at close spacing, so there is no need of providing any load transference, as this will be mainly done by aggregate interlocking.
- For added safety, use of dowel bar which are fully bonded in the concrete is recommended.

Warping Joints

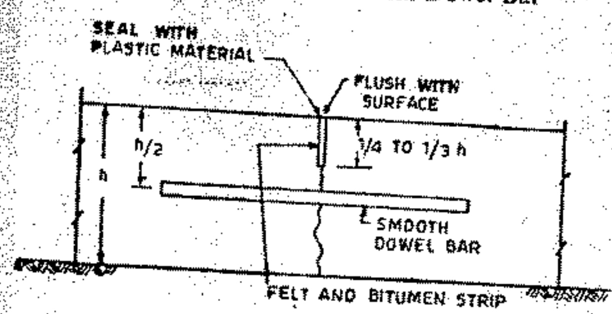
- Warping joints are provided to relieve stresses due to warping. These are known as *hinged joints.* Longitudinal joints with tie bars fall in this class of joint.
- These are rarely needed if suitably designed expansion and contraction joints are provided to prevent cracking.



(a) Dummy Joint



(b) Contraction Joint with Dowel Bar



(c) Contraction Joint with Dowel Bar

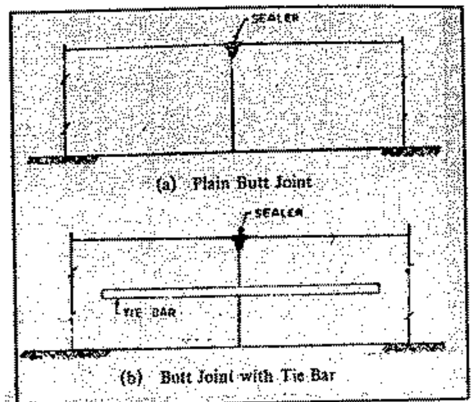
Contraction joints

Construction Joints

These joints are provided at the end of a day's work or when the work is stopped unexpectedly due to interruption for more than 30 minutes. These are either contraction joints or expansion joints

Longitudinal Joints

- Longitudinal joints are provided in cement concrete roads which have width over 4.5 m.
- On soil subgrade of clay, these are provided to allow differential shrinkage and swelling due to rapid changes in subgrade moisture under the edges than under the centre of road.
- Provided to prevent longitudinal cracking in the cement concrete pavements.
- Acts as a hinge and helps to maintain the two slabs together, at the same level.
- IRC recommends plain butt with tie bar type of joints.
- In slabs of thickness 20 cm, (a) 10 mm dia. deformed tie bars of length 35 cm or plain bars of length 45 cm are placed at 45 cm spacing (b) 12 mm dia. deformed bars of length 40 cm or plain bars of length 55 cm are placed at 64 cm spacing.



(a) Plain Butt Joint
(b) Butt Joint with Tie Bar

- In slabs of thickness 25 mm, tie bars of dia. 10, 12 or 14 mm and length 35-46 cm are placed at 30 to 62 cm spacing.

Joint Filler and Sealer

- The infiltration of water damages the soil subgrade and gives rise to the phenomenon known as *mud pumping* especially if the subgrade is of clayey soil.
- If stone grit enters into the joint space, the effective joint width gets reduced and faults like *spalling* of joint edges take place.

Joint Filler

Joint filler should possess the following properties :

(a) Compressibility (b) Elasticity (c) Durability

Types of Joint Filler

Following are the few types of joint filler materials :

(a) Soft wood (b) Impregnated fibre board (c) Cork or cork bound with bitumen

Soil Stabilisation

- Soil stabilization is a process of treating a soil in such a manner as to maintain, alter or improve the performance of the soil as a construction material.
- The changes in the soil properties are brought about either by the incorporation of additives or by mechanical blending of different soil types.

Purpose of Soil Stabilization

Soil-stabilization is practised in road construction with following objectives:

(i) To improve the strength of sub-bases, bases and, in the case of low-cost roads, surface courses.

(ii) To bring about economy in the cost of a road.

(iii) To make use of locally available soils and other materials which are otherwise inferior.

(iv) To eliminate or improve certain undesirable properties of soils, such as excessive swelling or shrinkage, high plasticity, difficulty in compacting etc.

(v) To control dust.

(vi) To facilitate compaction and increase load-bearing property.

(vii) To reduce frost susceptibility.

(viii) To reduce compressibility and thereby settlements.

(ix) To improve permeability characteristics.

Soil Stabilization Methods

The methods of soil stabilizatin which are in common use are :

(i) Mechanical soil stabilization
(ii) Soil-cement stabilization
(iii) Soil-lime stabilization
(iv) Soil-bitumen stabilization

Mechanical soil stabilization

- Correctly proportioned materials (aggregates and soils) when adequately compacted to get a mechanically stable layer, the method is called mechanical stabilization.
- Thus the two basic principles in this method of stabilization are :
  (i) Proportioning
  (ii) Compaction

Soil-cement Stabilization

- An intimate mix of soil, cement and water which is well compacted to form a strong base course.
- In granular soil, the mechanism of stabilization is due to the development of bond between the hydrated cement and the compacted soil particles at the points of contact.
- In fine grained soil, the stabilization is due to reduction in plasticity and formation of matrix enclosing small clay lumps.

Soil-lime Stabilization

- Soil-lime has been widely used either as a modifier for clayey soil or as a binder.
- When clayey soils with high plasticity are treated with lime, the plasticity index is decreased and the soil becomes friable and easy to be pulverised, having less affinity with water.
- Lime also imparts some binding action even in granular soils.
- Suitable as sub-base course for high types of pavements & base course for pavements with low traffic.

Stabilization of Black Cotton Soils

- Black Cotton (BC) soils are highly clayey soils, greyish to blackish in colour found in several states in India.
- The black cotton soils are found to contain *montmorillonite* clay mineral which has high expansive characteristics.

- The most effective method to stabilize BC soils are by using lime along with suitable additives;
- The cement requirement for satisfactory stabilization of BC soil is so high (15 to 25 percent) that it is not advisable to use portland cement, all by itself, for stabilization.

Stabilization of Desert Sand

- The desert sand deposits consist of fine grained uniformly graded sand with, rounded particles.
- The cement requirement for satisfactory stabilization is also very high in such soil.
- Due to the scarcity of water, soil-cement-stabilization is all the more difficult as considerable water is needed for soil cement base course construction.
- Keeping all these problems in view bituminous stabilization seems to be a suitable solution.
- The most promising bituminous material in desert region seems to be the emulsion.

OBJECTIVE QUESTIONS

1. In which one of the following bituminous constructions, compaction by pneumatic roller also is specified?
   (a) Premix carpet (b) Bituminous macadam
   (c) Bituminous concrete (d) Surface dressing

2. **Assertion (A):** The prime coat is an interface bituminous treatment when the existing base course has a pervious texture like water bound macadam.
   Reason (R): The primer has to get into the capillary voids in the existing base and it should be of low viscosity. Bituminous emulsion is generally used as a prime coat.

3. Consider the following stages in the construction of concrete roads:
   1. Preparing the subgrade and the base course.
   2. Mixing and placing the concrete.
   3. Placing the framework and watering the prepared base.
   4. Curing.
   5. Compaction and floating.

   The correct sequence of these stages is
   (a) 1, 2, 3, 4, 5 (b) 1, 3, 2, 5, 4
   (c) 1, 2, 3, 5, 4 (d) 1, 3, 2, 4, 5

4. Match **List-I** (Type of construction) with **List-II** (% Bitumen content) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Bituminous macadam	**1.** 3-3.5%
B. Dense bituminous macadam	**2.** ≤ 4%
C. Bituminous concrete	**3.** 14-17%
D. Bituminous mastic	**4.** Min. 4.5%

Codes:

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

5. Consider the following statements:
   1. Mastic asphalt is a mixture of hard grade bitumen or blown bitumen, minerals filler and fine aggregates.
   2. % of binder content in the mastic asphalt is 17-20 per cent by weight of the aggregates.

   Which of these statements is/are correct?
   (a) 1 only (b) 2 only
   (c) Both 1 and 2 (d) Neither 1 nor 2

6. Which one of the followings is the set of physical requirements of coarse aggregates for construction of WBM roads as per IRC recommendation?

	LAV(%)	AIV(%)	FI(%)
(a)	<50	<40	<15
(b)	<50	<30	<15
(c)	<40	<30	<20
(d)	<40	<30	<15

7. Match **List-I** (Type of construction) with **List-II** (Binder content) and select the correct answer using the code given below the lists:

List-I	List-II
A. Bituminous macadam	**1.** 8-15%
B. Dense bituminous macadam	**2.** 3-3.5%
C. Bituminous concrete	**3.** 4-4.5%
D. Bitumen mastic	**4.** 4.5-6.0%

Codes:

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

8. Consider the following bituminous surfacings:
1. SDBM
2. PMC
3. AC 4.
SD
5. Mastic Asphalt (MA)

Which one of the following is the correct sequence in increasing order with respect to their performance and wearing qualities?

(a) 4, 1, 2, 3, 5 (b) 2, 4, 1, 5, 3
(c) 4, 2, 1, 3, 5 (d) 1, 4, 3, 2, 5

9. Which one of the following is not a desirable property of the subgrade soil as a highway material?

(a) Stability (b) Ease of compaction
(c) Good drainage (d) Bitumen adhesion

10. Hot bitumen is sprayed over freshly constructed bituminous surface followed by spreading of 6.3 mm coarse aggregates and rolled. Which one of the following is indicated by this type of construction?

(a) Surface dressing (b) Gravel-bitumen mix
(c) Liquid seal coat (d) Seal coat

11. **Assertion (A):** In water-bound macadam construction, grade 1 has better load dispersion characteristics as compared to grade III aggregates.

Reason (R): The plasticity index of the binding material should be less than 6%.

12. Bituminous concrete is a mix comprising of
   (a) fine aggregate, filler and bitumen
   (b) fine aggregate and bitumen
   (c) coarse aggregate, fine aggregate, filler and bitumen
   (d) coarse aggregate, filler and bitumen

13. Consider the following statements:
   Selection of the type and grade of the bituminous binder is made on the basis of
   1. cost of the binder material
   2. climatic conditions of the area and the construction technique
   3. degree of importance of the road
   4. thickness of pavement layer to be constructed.

   Which of these statements is/are correct?
   (a) 1 only (b) 2 only
   (c) 1, 2 and 4 (d) 1, 2 and 3

14. Consider the following statements with reference to Water Bound Macadam (WBM) and Wet Mix Macadam (WMM):
   1. WBM is a road mix and WMM is a plant mix.
   2. WBM usually has plastic filler, while WMM has non-plastic filler.
   3. WBM is a modern road mix and WMM is a traditional road mix.

   Which of these statements is/are correct?
   (a) 1 and 2 (b) 2 and 3
   (c) 1 only (d) 2 only

15. Consider the following statements regarding soil stabilization:
   1. Subgrade should be treated if it has a soaked CBR of 5 or less.
   2. Sub-base, after treatment, should have plasticity index not less than 5.
   3. Base courses after treatment should have plasticity index of not more than 2 and a CBR in excess of 100.

   Which of these statements is/are correct?
   (a) 1, 2 and 3 (b) 1 and 3
   (c) 2 and 3 (d) 1 alone

16. With addition of lime in soil
   (a) LL increases and PL decreases
   (b) Plasticity Index increases
   (c) LL change very slightly and PL increases
   (d) LL and PL both decrease

ANSWERS

1.	(a)	2.	(c)	3.	(b)	4.	(b)	5.	(a)	6.	(a)	7.	(c)
8.	(c)	9.	(d)	10.	(a)	11.	(d)	12.	(c)	13.	(d)	14.	(a)
15.	(b)	16.	(c)										

CHAPTER

7

Highway Maintenance

INTRODUCTION

- By early detection & repair of defects at initial stages the rapid deterioration of the pavement can be prevented. Such surveys & evaluations should be carried out periodically so as to plan necessary preventive maintenance measures.

Maintenance of highway

Various maintenance operations are:

(1) **Routine maintenance :** These includes filling up of pot holes and patch repairs, maintenance of shoulders and the cross slope and repairing of cracks which are required to be carried out by the maintenance staff almost round the year.

(2) **Periodic maintenance :** These include renewals of wearing course of pavement surface and preventive maintenance of various items.

(3) **Special repairs :** These include major restoration or upgrading of the pavement through reconstruction or application of overlays to rectify structural deficiencies.

Symptoms, Causes, And Treatment of Defects

The types of defects in bituminous surfacing are grouped under four categories :

(i) **Surface defects :** which include fatty surfaces, smooth surfaces, streaking, and hungry surfaces;

(ii) **Cracks :** under which hair-line cracks, alligator cracks, longitudinal cracks, edge cracks, shrinkage cracks, and reflection cracks are dealt with ;

(iii) **Deformation :** under this are grouped slippage, rutting, corrugations, shoving, shallow depressions, and settlements and upheavels; and

(iv) **Disintegration :** covering stripping, loss of aggregates, ravelling, pot-holes, and edge breaking.

- We will 1st of all describes the symptoms and causes of these defects and indicates the possible types of treatment.
- In each case of pavement distress, the cause or causes of the distress should first be determined. It will be possible to provide suitable maintenance measures which will not only correct the damage but also prevent or delay its recurrence.
- In many situations, lack of proper drainage is the principal cause for stripping loss of materials from the pavement and shoulder, weakening of the pavement layers and subgrade, resulting in the failure of the pavement.

SURFACE DEFECTS

These are associated with the surfacing layers and may be due to excessive or deficient quantities of bitumen in these layers.

(1) Fatty Surface

Symtoms

- Fatty surface, results when the bituminous binder moves upward in the surfacing and collects as a film on the surfae.
- The binder so colloected becomes generally soft in hot weahter and may be picked up and spread by the traffic. In cold or wet weather, the surface is likely to be slippery and this can cause accidents.

Causes

The causes for a fatty surface are :

(i) Excessive binder in a premix surfacing over-filling the voids.
(ii) Loss of cover aggregates in surface dressing.
(iii) Non-uniform spreading of cover aggregates in surface dressing.
(iv) Excessive application of binder in surface dressing.
(v) Poor quality of aggreages with leading to their fracture, breakdown and eventual loss.
(vi) Graded cover aggregates with particles so small that they are covered by the binder.

Treatment

(i) If the bleeding is fairly uniform and the surface is free from irregularities, application of cover aggregates or sand (sand-blotting or sand-blinding) would be successful. The aggregate or sand used shall be of small size, clean and angular, and may be heated, if necessary.
(ii) An open-graded premix surfacing with a low bitumen content can absorb the excess binder.
(iii) A liquid seal coat, with special care taken to select the rate of application of the binder and the quantity and size of cover aggregates, can also be effective.
(iv) Special methods such as the burning of the excess binder.
(v) In case of large areas of fatty surface having irregularities, removal of the affected layer in the area and replacing it with a layer having a properly designed mix, may be necessary.

(2) Smooth surface

Symtoms

A smooth surface, has a very low skid resistance value and becomes very slippery when wet. Such a condition invites hazards, especially on gradients, bends, and intersections.

Causes

A primary cause for a smooth surfae is the polishing of aggregates under traffic. Also excessive binder can result in a smooth surfae

Treatment

The recitification consists of resurfacing with a surface dressing course or a premix carpet. Care should be taken to select hard and angular aggregates which have proven non-plishing characteristics. The carpet can be an open-graded mix. A slurry seal can also be used to impart anti-skidding texture.

(3) Streaking

Symptoms

Streaking is characterised by the appearnace of alternate lean and heavy lines of bitumen either in longitudinal, or in transverse direction.

Causes

- Longitudinal streaking results when alternate longitudinal strips of surface contain different quantities of bitumen due to non-uniform application of bitumen across the surface.
- Some of the more common causes of this type of streaking are mechanical faults, improper or poor adjustment and careless operation of bitumen distributors.
- These streaks can also be formed as a result of applying the bituminous binder at too low a temperature; a temperature at which bitumen is not fluid enough to fan out properly from the nozzles on the spray bars.
- All these causes can result in transverse streaking also. Transverse streaking may also be caused by spurts in the bitumen spray from the distribution spray bar.
- Transverse streaking may result in corrgation in the pavement surface.

Treatment

- The satisfactory repair for longitudinal and transverse streaxing is to remove the streaked surface and apply a new surface treatment.
- It is always desirable to prevent longitudinal and transverse reaking than to correct it.

(4) Hungy Surface

Symptoms

Hungry surface is characteristcharacterised by the loss of aggregates from the surface or the appearnace of fine cracks.

Causes

- One of the reasons for hungry surface is the use of less bitumen in the surfacing. Sometimes this condition may also appear due to use of absorptive aggregates in the surfacing.

Treatment

A slurry seal may be used as a repair meausre. It is applid in an average thickenss of 2 – 5 mm.

- As an emergency repair, a fog seal may be used.

CRACKS

General : A common defect in bituminous surfaces is the formation of cracks.

- The crack pattern can in many caes, indicate the cause of the defect.
- As soon as cracks are observed, it is necessary to study the pattern in detail so as to arrive at the cause.
- Immediate remedial action should be taken thereafter because of the danger of ingress of water through the cracks and of the formation of pot-holes and revelling.
- Cracks can hardly be observed from moving vechiles and inspection on foot is always desirable.
- The common types of cracks are discussed breifly in the following clauses.

(a) Hair-line Cracks

Symptoms

These appear as short and fine cracks at close intervals on the surface,

Causes

These cracks are caused by :

(i) Insufficient bitumen content.

(ii) Excessive filler at the surface.

(iii) Improper compaction – over-compaction, compaction when the supporting layer was unstabe, or compaction of too hot a mixture.

(b) Alligator Crack

Symptoms

These appear as interconnected cracking forming a series of small block which resemble the skin of an alligator. This pattern is also called map cracking.

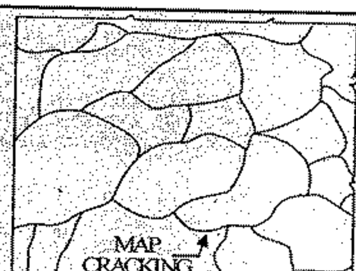


Fig. Map Cracking

Causes

Alligator cracks are due to one or more of the following factors :

(i)) Excessive deflection of the surface over unstable subgrade, sub-base or base of the pavement, particularly in the wheel tracks. The unstable conditions in the subgrade or lower layers of the pavement might have arisen from saturation.

(ii) Excessive overloads by heavy vechiles or inadequate pavement thickness, or both.

(iii) Brittleness of the binder either due to ageing of binder or initial over-heating might cause fine cracks of the alligator pattern, but there will be no deflection of the surface. These cracks are sometimes called 'crazing'.

(c) Longitudinal Crack

Symptoms

These cracks appear, more or less, on a straight line, along the road. These cracks may appear either at the joint between the pavement and the shoulder, or at the joint between two paving lanes.

Causes

(i) The cracking at the pavement-shoulder joint may be due to alternate wetting and drying beneath the shoulder surface owing to poor drainage or due to depressions in the pavement edge which allow water to stand and seep through the joint. Shoulder settlement or trucks passing over the joint, may also cause these cracks.

(ii) The lane joint crack is caused by a weak joint between adjoining spreads in the layers of the pavement. Differential 'frost heave along the centre line may also be one of the caues'.

(d) Edge Crack

Symptoms

Edge cracks are formed parallel to the outer edge of the pavement usually 0.3 – 0.5 m inside from the edge, at times some transverse cracks are seen to branch out from the edge cracks towards the shoulder.

Causes

These cracks ar caused by :

(i) Lack of lateral support from the shoulder.

(ii) Settlement or yielding of the underlying material.

(iii) Inadequate surface drainage, especially during flooding conditions.

(iv) Shrinkage due to drying out of the surrounding earth, generally caused by roots of trees or bushes close to the pavement edge. Highly expansive soils are particularly prone to shrinkage when moisture dries out.

(v) Frost heave.

(vi) Inadequate pavement width forcing traffice too close to the edge of the pavement.

(vii) Non-provision of extra width of pavement on curves.

(e) Shrinkage Crack

Symptoms

These are cracks appearing in the transverse direction, or as interconnected cracks forming a series of large blocks. The pavement itself appears to have suffered no deterioration or deformation, but it is the top surfacing that seems to have become old and cracked.

Cause

The primary cause for such cracks is the shrinkage of the bituminous layer itself with age. The bituminous binder loses its ductility as it ages and becomes brittle.

(f) Reflection Crack

Symptoms

Reflection cracks are the sympathetic cracks that appear in the bituminous surfacing over joints and cracks in the pavement underneath. the pattern may be longitudinal, transverse, diagonal or block.

- They occur most frequently in overlays on cement concrete pavements or on cement-soil bases.
- They may also occur in overlays or surfacings on flexible pavements where cracks in the old pavement have not been properly repaired.
- Another condition under which reflection cracks can occur is when a pavement is widened and the entire pavement is surfaced.
- The location of the crack will then be exactly on the junction between the old pavement and the widened strip.
- In some cases reflection cracks are merely unsighly, but frequently they deteriorate and the riding quality of roads is affected.
- These cracks can allow water to enter the underlying pavement and the subgrade and cause further damage.

Cause

Reflection cracks are due to joints and cracks in the pavement layer underneath.

Treatment

The treatment, for all types of cracks discussed above, would depend on whether the pavement remains structurally sound, or has become distorted or unsound.

- In case the pavement remains structurally sound, then the cracks should be filled with a bituminous binder having a low viscosity so that it can be poured and worked into the cracks.
- Cut-back bitumen and emulsions are generally suitable.
- All loose materials are removed from the cracks with brooms and, if necessary, with compressed air jetting.
- The binder is poured with a pouring can and a hand squeegee is used to assist the penetration of the binder into the cracks.
- Light sanding of the cracks is then done to prevent traffic picking up the binder.
- If the cracks are wide enough a slurry seal or sand bituminous premix patching can be used to fill the cracks.
- If the cracks are fine (crazing) and extend over large areas, a light cut-back or an emulsified bitumen (for seal) can be broomed into the cracks and lightly sanded to prevent the picking up of the binder by the traffic.

DEFORMATION

General : Any change in the shape of the pavement from its original shape is a deformation. It may be associated with slippage, rutting, etc., discussed below. The treatment measures aim at the removal of the cause, and bringing it to the original level by fill material or by removing the entire affected part and replacing it with new material.

(a) Slippage

Symptoms

- Slippage is the relative movement between the surface layer and the layer beneath.
- It is characterised by the formation of crescent-shaped cracks that point in the direction of the thrust of the wheels on the pavement surface.
- This does not mean that the cracks invariably point in the direction the traffice is going.
- For example, if breakes are applied on a vehcile going down a hill the thrust of the wheels will be pointing uphill. the cracks in this case will, therefore, point uphill.

Causes

Slippage is caused by :

(i) Unusual thrust of wheels in a particular direction.

(ii) Omission or inadequacy of tack coat or prime coat.

(iii) Lack of bond between the surface and the lower course caused by a layer of fine dust, moisture or both.

(iv) Failure of bond between two layers due to excessive deflection of the pavement.

Treatment

Rectification consists of removing the surface layer around the area affected upto the point where good bond between the surfacing and the layer underneath exists and patching the area with premix material after a tack coat.

(b) Rutting

Symptoms

- Rutting is a longitudinal depression or groove in the wheel. The ruts are usually of the width of a wheel path. Swerving from a rutted wheel path at high speed can be dangerous. Accumulation of water in the depressions can cause skidding. If rutting is accompanied by adjacent bulging, it may be a sign of subgrade movement or weak pavement.

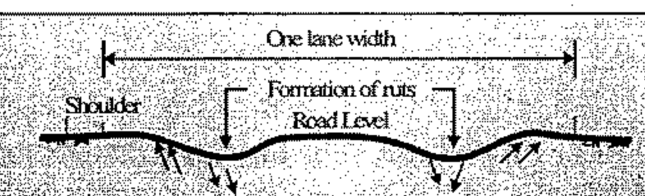


Fig. Formation of Ruts

Cause

The cause of rutting are the following :

(i) Heavy channelised traffic.

(ii) Inadequate compaction of the mix at the surface or in the under lying courses during construction.

(iii) Improper mix design, lacking in stability of the mix to support the traffic and leading to plastic movement laterally under traffic.

(iv) Weak pavement.

(v) Incidence of high stress caused by heavy bullock cart traffic.

(vi) Intrustion of subgrade clay into base course.

(vii) Aggregates of surface dressing being pressed into the lower supporting bituminous layer.

Treatment

The rectification consists of filling with premix open-graded or dense-graded patching materials and compacting to the desired levels. The limits of the depression are first determined with a string line and marked on the surface. After applying a suitable tack coat, the premix is spread and compacted.

Situations indicative of shear failure or subgrade movement generally require excavation. The job should be carefully assessed. The area to be opened up should as far as possible be limited to that which can be completed and made safe in a day's working.

(c) Corrugation

Symptoms

- Corrugation is the formation of fairly regular undulations (ripples) across the bituminous surface. They are usually shallow (about 25 mm) and are different from the larger depressions caused by weakness in the lower layers of the pavement of the subgrade. The spacing of the waves is around 3 m. The corrugations can be a source of discomfort to the motorists and can become a hazard if allowed to become severe.

Causes

Corrugations are due to the following causes :

(i) Lack of stability in the mix (excessive binder, high proportion of fines, too round or too smooth textured coarse or fine aggregate, too soft a binder).

(ii) Oscillations set up by the vehicle springs can cause alternative valleys and ridges.

(iii) Faulty laying of surface course.

Treatment

If the surface is thin, the same is scarfieid, including some portions of the underlying water-bound macadam base, and the scarified material is recompacted. A new surfacing layer is then laid.

Cutting of high spots with a blade with or without heating and addition of levelling course materials can also be an effective way to make up the corrugations. The area is then thoroughly rolled.

(d) Shoving

Symptoms

Shoving is a form of plastic movement within the layer resulting in localised bulging of the pavement surface. Shoving occurs characteristically at points where traffic starts and stops (intersections, busy bus-stops), on hills where vehicles accelerate or brake on grades and on sharp curves. The first indication of shoving is the formation of slippage cracks which are crescent shaped cracks with the apex of the crack pointing in the direction of the shove.

Cause

Shoving can be caused by :

(i) Lack of stability in the mix (excessive binder, high porportion of fines, too soft a binder) in the surface or base course.

(ii) Lack of bond between bituminous surface and underlying layer.

(iii) Heavy traffice movement of a start and stop type or involving negotiation of curves and gradients.

(iv) Use of non-volatile oil on roller wheels.

Treatment

The rectification consists of removing the material in the affected area down to a firm base and laying a stable premix patch.

(e) Shallow depresion

Symptoms

Shallow depressions are localised low areas of limited size, dipping about 25 mm or more below the desired profile, where water will normally collect. The depressions may or may not be accompanied by cracking. If not recitifed in time, they may lead to further deterioration of the surface and cause discomfort to traffic.

Cause

Shallow depressions are caused by the settlement of lower pavement layers due to a pocket of inadequately compacted subgrade or pavement layers.

Treatment

Shallow depression are made up by filling with premix materials, open-graded or dense graded, and compacting to the desired profile as the surrounding pavement.

(f) Settlement and upheaval

Symptoms

Settlement and upheavals are characterised by large deformations of the pavement. They are extremely uncomfortable to traffic and cause serious reduction in speed. They are generally followed by extensive cracks in the pavement surface in the affected region.

Causes

The following are the causes for settlements and upheavals :

(i) Inadequate compaction of the fill at locations behind bridge abutments, over utility cuts, etc.

(ii) Excessive moisture in subgrade and permeable layer of sub-base and base caused by capillary action or poor drainage.

(iii) Inadequate pavement thickness.

Treatment

If settlement and upheavals indicate an inherent weakness in the fill, it may be necessary to excavate the defective fill and do the embankment afresh under properly controlled conditions. Under-drains may become necessary in locations where lack of drainage has been identified as the cause of failure. Where the cause of deformation is inadequate pavement thickness, then properly designed pavement shall be provided. Frost-affected regions may need through investigations and a complete reconstruction of the pavement.

DISINTEGRATION

General : These are some defects which if not rectified immediately, result in the disintegration of the pavement into small, loose fragments. Disintegration, if not arrested in the early stages, may necessitate complete rebuilding of the pavement.

(a) Stripping

Symptoms

This defect is characterised by the separation of bitumen adhering to the surfaces of the aggregate particles, in the presence of moisture. This may lead to loss of bond and subsequently to loss of strength and materials from the surface.

Causes

Stripping may be caused by the following :

(i) Use of hydrophilic aggregates.

(ii) Inadequate mix composition.

(iii) Continuous contact of water with the coated aggregate.

(iv) Initial over-heating of the binder or the aggregate or both.

(v) Presence of dust or moisture on aggregate when it comes in contact with the bitumen.

(vi) In the case of surface dressing, poor bond with the surface existing below, delay in spreading the cover aggregate over the sprayed bitumen, or insufficient compaction.

(vii) Occurrence of rain or dust storm immediately after the construction.

(viii) Opening the road to fast traffice before the binder has set.

(ix) Concentration of soil salt in rain water coming in contact with the coated aggregate.

(x) Use of improper grade of bitumen.

(xi) Ageing of the bitumen leading to the embrittlement of the binder film.

Treatment

In the case of surface dressing, hot coarse and heated to at least 159°C and spread over the affected areas, may be used to replace the lost aggregates. After spreading, it should be rolled immediately so that it will be seated into the bitumen. If aggreages are only partially whipped off, a liquid seal may be the solution.

In other cases the existing bituminous mix should be removed and a fresh one laid. As a precautionary measure, a suitable anti-stripping agent should be added to the bitumen, at the time of construction.

(b) Loss of aggregate

Symptoms

Loss of aggregate occurs in surfaces which have been provided with surface dressing. The surface presents a round appearance, with some portions having aggregates intact and others where aggregates have been lost,

Causes

The loss of aggregates can occur due to the following causes :

(i) Ageing and hardening (oxidation) of the binder whereby its adhesive property is lost.

(ii) Stripping of binder from aggregates due to cold or wet weather before, during or soon after surface dressing.

(iii) Wet or dusty aggregate to which binder has not adhered.

(iv) Insufficient binder for the size of the aggreagte used or for the existing absorptive surface.

(v) Aggregate having no affinity to the binder.

(vi) Insufficient rolling before opening to traffic.

(vii) Fast traffic over new work whipping off the aggregates.

(viii) Cold-spraying of bitumen or delaying the spreading of aggregates over sprayed bitumen.

Treatment

If the loss of aggregates is due to ageing and hardening of the binder, the condition may be rectified by applying liquid seal, fog seal or slurry seal.

If the loss of aggregates has occurred over large isolated areas, the best thing to do would be to provide another surface dressing layer, after carefully cleaning the surface.

If the loss of aggregates has taken place in small isolated patches a liquid seal would be sufficient.

(c) Ravelling

Symptoms

Ravelling is generally associated with premixed bituminous layers. It is characterised by the progressive disintegration of the surface due to the failure of the binder to hold the materials together. The ravelling process generally starts from the surface downwards or from edge inward. It usually begins with the blowing off of the fine aggregates leaving behind pock marks on the surface. When larger particles are broken free, the surface appears eroded.

Causes

Ravelling is due to one or more of the following reasons :

(i) Inadequate compaction during construction.

(ii) Construction during wet weather leading to stripping of binder from aggregates.

(iii) Construction during cold weather resulting in non-uniform binder film.

(iv) Use of inferior quality aggregate resulting in fracture, crushing and opening of new faces.

(v) Insufficient binder in the mix.

(vi) Ageing of binder leading to brittle fracture and disintegration of pavement.

(vii) Excessively open graded mix.

(viii) Poor compatibility of binder and aggregate.

(ix) Over-heating of mix or the binder.

(x) Improper coating of aggregates by binder.

Treatment

Ravelled surface is corrected by adding more quantity of binder, the rate of application depending upon the condition of existing surface and degree of hardening occurred to the binder. If the ravelling has not developed too far, the condition may be corrected by a simple application of a cut-back bitumen covered with coarse sand, or a slurry seal can be applied. Where the ravelling has progressed far, a renewal coat with premix material would be necessary.

(d) Pot-hole

Symptoms

Pot-holes are bowl-shaped holes of varying sizes in a surface layer or extending into the base course caused by localised disintegration of material. They usually appear after rain.

Causes

(i) The most common cause of pot-hole formation is the ingress of water into the pavement through the surfacing course. This can happen if the surfacing is open-textured and lacks proper camber. Water can enter the pavement also through the cracks in the bituminous surface. The pavement gets softened as a result, and under the action of traffic a depression soon gets formed. This is aggravated by use of plastic filter in WBM. If not attended to properly, the aggregates in the surfae get progressively loosened and a regular pot-hole forms.

(ii) Lack of proper bond between the bituminous surfacing and the underlying water bound macadam base can also cause pot-holes. The bond is usually supplied by a tack coat, and any localised inadequacies in these applications can cause pot-holes.

(iii) Insufficient bitumen content in localised areas of the surfacing layer can cause pot-holes.

(iv) Too thin a bituminous surface which is unable to withstand the heavy traffic can also cause pot-holes, when associated with improper or inadequate camber.

(v) In dense-graded mixtures, pot-holes can be caused by too much fines or too few fines.

Treatment

the rectification consists of filling pot-holes with premix open-graded or dense-graded patching, or penetration patching.

(e) Edge-breaking (Frayed edges)

Symptoms

A common defect in bituminous surfaced roads is edge breaking. The edge of the bituminous surface gets broken in an irregular way, and if not remedied in time, the surfacing may peel off in large chunks at the edges.

Causes

The following are the causes for edge breaking :

(i) infiltration of water which softens the foundation layers causing the pavement edges to break.

(ii) Worn out shoulders resulting in insufficient side support to the pavement.

(iii) Inadequate strength at the edge of the pavement due to inadequate compaction.

(iv) Lower layer of pavement not being wider than upper layer.

Treatment

- The shoulder and the pavement material in the affected area should be entirely removed to a regular section with vertical sides.
- The pavement and the shoulders should be built up simultaneously with thorough compaction.
- A bituminous surface similar to that in the adjacent reach should be laid.
- The shoulder should have adequate slope to drain away the water.
- A slope one per cent steeper than the camber of the bituminous surface should be found generally necessary for earthen shoulders.
- In order to prevent the edges from getting broken again, the maintenance operations should include periodic inspection of the shoulder condition and replacment of worn out shoulder material with adequate compaction.
- In sandy areas where the soil is likely to be eroded by wind and rain, it may be advantageous to have brick paving at least for some width to protect the edges. Surface and subsurface drainage, wherever deficient, should be improved.

METHODS FOR REPAIRING THE DEFECTS

- The repair methods discussed in this section generally fall under two categories: (i) Seal Coat, and (ii) Patching.
- The seal coat, is a single, thin application of bitumen which may or may not be covered with aggregate.
- Patching is the application of bituminous materials either premixed or penetration macadam type, and is resorted to for filling pot-holes, shallow depressions, rutting and edge irregularities.
- Patching when used for filling the rutting and depressions, can also be termed as "levelling".

Liquid Seal

Description : Liquid seal consists of an application of liquid bitumen (penetration grade, cut-back or an emulsion) and covering the same with aggregate. this is applicable for the rectification of fatty surfaes, stripping, loss of aggregates, and ravelling.

Materials : Penetration grade bitumen should be of suitable grade. A cold application cut-back such as RC-3 or MC-3 is also suitable. If emulsion is to be used, it should be of the rapid setting type. The quantity of binder shall be as follows :

Blinder	Quantity for 10 sq. m (kg.)
1. Penetration grade bitumen	9.8
2. Cold application cut-back, RC-3 or MC-3 (Quantity in terms of penetration grade bitumen)	9.8
3. Emulsion, RS	10–12

The cover aggregates should be a noninal size of 6.3 mm viz. passing through 10 mm IS Sieve and retained on 2.36 mm IS Sieve. The quantity of cover aggregate should be 0.09 cu. m per 10 sq. m.

Fog Seal

Description : Fog seal is a light application of emulsified bitumen, usually without a cover aggregate. It is used to increase the binder content of bituminous surfaces, rejuvenate oxidised and old surfaces, fill in cracks and prevent ravelling. It can also be used as an emergency treatment measure for hungry surfaces.

Slurry Seal

Description : Slurry seal is a mixture of fine aggregates, mineral filler and emulsified bitumen with water added to achieve slurry consistency. The ingredients are mixed and spread evenly on to bituminous surfaces to fill cracks, repair ravelled pavements, smooth or hungry surfaces, rectify loss of aggregates, rejuvenate oxidised and open-textured old bituminous surfaces, and to provide a skid resistant surface.

Materials : The aggregate gradation is very important. The following grading is found suitable :

Gradation of aggregation for slurry seal

Sieve designation	Per cent by weight passing the sieve
4.75 mm	100
2.36 mm	80-100
1.18 mm	50-90
300 micron	15-50
150 micron	10-25
75 micron	3-10

In order to obtain the above gradation, fine grit, sand and filler can be mixed in suitable proportions.

The binder is a slow-setting emulsified bitumen. The mix has to be designed to have a consistency such that the slurry when spread,would flow in a wave approximately half a metre ahead of the strike-off squeegee. This would ensure that the slurry would not bridge over the crakcs without filling them. About 18–20 per cent emulsion and 10–12 per cent of water by weight of the aggregates would approximately make a satisfactory mix.

Sand Bituminous Premix Patching

Description :

- Sand bituminous premix patching consists of laying a mixture of fine aggregate and bituminous binder to rectify cracks, slippage, corrugations, shoving, shallow depressions and ravelling.
- The fine aggregate shall be a medium coarse sand (fineness modulus of more than 2.5) or fine grit passing 1.70 mm IS Sieve and retained on 180-micron IS Sieve. The binder can be a paving bitumen of suitable penetration grade, rapid curing cut-back such as RC-3, or a medium curing cut-back such as MC-3.

Premix Open-graded Patching

Description : Premix open-graded patching consists of making up the area to be patched by a premix open-graded material consisting of a binder and aggregates, compacting and finishing with a seal coat. This repair method is applicable for fatty surfaces, slippage, rutting, shoving, shallow depressions and pot holes.

Materials : Stone aggregates of the following sizes and at the quantities specified below shall be used for premixing:

	Quantity for 10 sq. m. area (For a thickness of 20 mm)
(i) Coarse aggregates–12.5 mm size (passing 20 mm IS Sieve and retained on 10 mm IS Sieve).	0.18 cu.m.
(ii) Coarse aggregates–10 mm size Passing 12.5 mm IS Sieve and retained on 6.3 mm IS Sieve).	0.09 cu. m.
Total :	0.27 cu m

The binder can be paving bitumen of suitable penetration grade, rapid curing cut-back such as RC-3 medium curing cut back such as MC-3, or a medium setting bitumen emulsion. The quantities of binder for various operations are indicated below :

	Quantity for 10 sq. m. area (kg.)	
	Tack coat	**Premix** (for a thickness of 20 mm)
1. Penetration grade bitumea	7.5	14.6
2. Cold application cut-back, RC-3 or MC-3 (Quantity in terms of penetration grade bitumen)	7.5	14.6
3. Emulsion, MS	7.5	15-18

Premix Dense-graded Patching

Description :

- Premix dense-graded patching consists of making up the area to be patched by a dense-graded premix material consisting of a binder, aggregates and filler, compacting and finishing.
- This is a high quality, thoroughly controlled hot mixture for which the mix design is to be invariably got done before the start of the work in a suitably equipped laboratory.
- For existing superior types of surfaces, the use of this type of patching may be considered.
- This type of patching can be used for prepairing slippage, rutting, shoving, shallow depressions, or pot-holes.

Materials :

- The coarse aggregates, fine aggregates and filter shall be mixed in suitable proportions.
- The binder shall be a paving bitumen of suitable penetration grade. the quantity of binder by weight of total mix shall be 5.7.5 per cent.

Design requirements : The design requirements of the mix shall be as under :

Number compaction blows, on each	
end of Marshall specimen	50
Marshall stability in 1b.	750
Marshall Flow-0.01 in	8-16
Per cent voids in mix	3-5
Percent voids in mineral aggregates	
filled with bitumen	75-85

Penetration Patching

Description :

- Penetration patching consists of making up the area to be patched by a course of aggregates, compacting the same, applying bitumen and key aggregates and finishing off with a seal coat. The patch is used for surface disintegration over 12 mm deep.
- The patching may be done in layers, but the depth of the individual layers should not exceed 50 m.

Although it is known from experience that penetration patching does not produce as good a patch as premix material due to lack of accurate control of the amount of bitumen to be used emergency patches of this type may be necessary as a last resort in the absence of premix material. Hence use of penetration patch should be very limited.

- The binder shall be a paving bitumen of suitable penetration grade, rapid curing cut back such as RC-3, or medium curing cut-back such as MC-3. The quantities of binder are indicated below

	Quantity for 10 sq. m. area (kg.)	
	Tack coat	Grouting
1. Penetration grade bitumen	7.5	50
2. Cold application cut-back, RC-3 or MC-3 7.5 (Quantity in terms of penetration grade bitumen)		50

CONSTRUCTION AND MAINTENANCE DURING WET WEATHER CONDITIONS

Nature of problem

It is well known that the chippings which are damp or wet due to storage in the open stacks, do not adhere to bituminous binder so long as the film of water persists on the stone. Also majority of wet weather failure of surface dressings are caused by rain during or within a few days after construction.

Maintenance measures

Use of bitumen emulsions

The bitumen emulsions not only flow well at atmospheric temperature but can also be applied to damp road surfaces and used for coating damp aggregates. the presence of emulsifying agent helps in improving the adhesion characteristics of the binder.

- Two types of bitumen emulsion namely the anionic type and the cationic type are generally used. The basic difference between the two lies in their method of breaking.
- Anionic bitumen emulsions break only when sufficient water has evaporated from the system to leave the emulsion unstable.
- With cationic bitumen emulsion the break is chemical, i.e., the positively charged emulsifier is chemically attracted to negative surface on the aggregate. This causes the emulsion to break and the emulsifier then acts as an adhesion agent.
- The cationic emulsifiers are especially useful with siliceous aggregate and may be effectively used in slurry sealing and preparation of patching mixtures.

MATERIALS

Binder for Maintenance work

The bituminous binder should be one of the following :

(1) Paving bitumen penetration grade 30/40, 60/70 or 80/100 conforming to IS: 73-1961.

(2) Cut-back bitumen of the Rapid curing (RC-3) and Medium curing (MC-3)

(3) Bitumen emulsions of the Rapid setting (RS) or Medium setting (MS) or Slow setting (SS) type.

The choice of a particular type of binder will depend upon the maintenance specification for which the binder is required, the climatic conditions, traffic and durability. The following broad indications may be tentatively taken as a general guide :

Type of Binder

A. Penetration Grade :

1. 30/40 Pen
(S 35)

Uses in Maintenance operations

(i) Premix dense-graded patching, penetration patching, penetration patching, premix open-graded patching/permix carpet, sand bituminous premix/premix seal coat, liquid seal, mix seal surfacing and semi-dense carpet.

(ii) Generally favoured in regions with compatively high atmospheric temperature throughout the year with slight variation between day and night temperatures.

(iii) Being a harder grade, recommended for heavy traffic, bus stops, parking places, etc.

2. **60/70 Pen**

(S 65)

(i) Premix dense-graded patching, penetration patching, premix open-graded patching/premix carpet, sand bituminous premix/premix seal coat, liquid seal, mix seal surfacing and semi-dense carpet.

(ii) Generally favoured for high summer temperatures and low winter temperatures, moderate variation between day and nigh temperatures.

(iii) Being a reasonably hard grade, preferred for heavy traffic, bus stops, parking places, etc.

3. **80/100 Pen**

(S 90)

(i) Premix dense-graded patching, penetration patching, premix open-graded patching/premix carpet, surface dressing, premix seal coat, liquid seal coat, mix seal surfacing and semidense carpet.

(ii) Generally favoured for areas with extremes in summer and winter temperatures and where the difference between day and night temperature is large ; and also for the low temperature regions at high altitudes.

B. Cut-backs:

1. **MC-3**

(Medium curing)

(i) Premix open-graded patching/premix carpet, semi-dense carpet, sand bituminous premix coat surface dressing, and liquid seal patching.

(ii) Generally favoured for maintenance operations where cold-mixing is an advantage.

(iii) Intended for immediate use after mixing as well as for stockpiling.

2. **RC-3**

(Rapid curing)

(i) Liquid seal, premix open-graded patching/premix carpet, surface dressing, sand patching and mix seal surfacing.

(ii) Generally forward for maintenance operation where cold mixing is an advantage.

(iii) Intended for immediate use after mixing.

C. Emulsions (anionic and cationic)

Emulsions may be the anionic, electro-negatively charged asphat globules, or cationic, electro-positively charged asphat globules, depending upon the emulsifying agent. Anionic emulsion combiens well with positive surface charged carbonate rocks, while cationic emulsions are more suitable with negative surface charged siliceous rocks.

1. Rapid setting :	(i)	Surface dressing, liquid seal.
	(ii)	Cationic emulsions can be used with wet aggregates (very useful for maintenance operations during rainy seasons).
2. Medium setting :	(i)	Premix open-graded patching
	(ii)	Cationic emulsions can be used with wet aggregates (very useful for maintenance operations during rainy seasons).
3. Slow setting :	(i)	Slurry seals, fog seals.

Stone Aggregates

The stone aggregates shall consist of crushed stone, crushed slag, crushed gravel (shingle) and sand and shall be clean, hard, tough durable and of uniform quality. They shall be free of elongated or flaky pieces, soft and disintegrated material, and organic or other deleterious matters. They should satisfy the following general requirements :

	Value	Method of Test
1.	Loss Angles A brasion value	Max. 35%
	or	
	Aggregate Impact Value	Max. 30%
2.	Flakiness Index	
	(i) Surface dressing and premix carpet	Max. 30%
	(ii) Bituminous macadam and asphaltic concrete	Max. 35%
3.	Stripping Value	Max. 25%
4.	Water absorption	Max. 2%
5.	Soundness : Loss with sodium sulphate –5 cycles (in case of slag only)	Max. 12%
6.	Unit weight or bulk density (in case of slag only)	Min. 1120 kg

FAILURE IN RIGID (CEMENT CONCRETE) PAVEMENTS

Failure of cement concrete pavements are perceived mainly by the formation of structural cracking. The failures are mainly due to two factors.

(i) Deficiency of pavement materials.

(ii) Structural inadequacy of the pavement system.

Deficiency of Pavement Materials

Following are the chief causes which would give rise to the different defects or failures of cement concrete pavement :

(i) Soft aggregates
(ii) Poor joint filler and sealer material
(iii) Poor surface finish
(iv) Improper and insufficient curing
(v) Poor workmanship in joint construction

The various defects which arise in due to the above are

(i) Disintegration of cement concrete
(ii) Formation of cracking
(iii) Spalling of joints
(iv) Poor riding surface
(v) Slippery surface
(vi) Formation of shrinkage cracks
(vii) Ingress of surface water and further progressive failures.

Structural Inadequacy of Pavement System

- Inadequate subgrade support or pavement thickness would be a major cause of developing structural cracking in pavements.
- Following are the causes of types of failure which develop :
  (i) Inadequate pavement thickness.
  (ii) Inadequate subgrade support and poor subgrade soil.
  (iii) Incorrect spacings of joints.

Above would give rise to the failures of the following types :

(i) Cracking of slab corners.
(ii) Cracking of pavements longitudinally.
(iii) Settlement of slabs.
(iv) Widening of joints.
(v) Mud pumping.

Typical Rigid Pavement Failures

Following are some typical and basic types of failures in rigid pavements

(i) Scaling of cement concrete
(ii) Shrinkage cracks
(iii) Spalling of joints
(iv) Warping cracks
(v) Mud pumping
(vi) Structural cracks

Scaling of Cement Concrete

- Scaling is observed in cement concrete pavement and shows overall deterioration of the concrete.
- The main reason behind scaling is the deficiency in the mix or presence of some chemical impurities which damage the mix.
- Due to excessive vibration given to mix, the cement mortar comes to the top during construction and thus with use, the cement mortar gets abraded exposing the aggregate of the mix. This makes the pavement surface rough and shabby in appearance.

Shrinkage Cracks

- Shrinkage cracks normally develop during the curing operation of cement concrete pavements immediately after the construction.
- The placement of cracks are both in longitudinal and transverse direction

Spalling of Joint

Sometimes when pre-formed filler materials are placed during casting of pavement slabs, the placement is some how dislocated and filler is thus placed at an angle. The concreting is completed without noticing this faulty alignment of the filler material. Thus this forms an overhang of a concrete layer on the top side and the joint later on shows excessive cracking and subsidence.

Warping Cracks

- If the joints are not well designed to accommodate the warping of slabs at edges, this results in development of excessive stresses due to warping and the slab develops cracking at the edges in an irregular pattern.
- Hinge joints are generally provided for relieving the slabs of warping stresses.

Mud Pumping

- Mud pumping is perceived when the soil slurry ejects out through the joints and cracks of cement concrete pavement caused during the downward movement of slab under the heavy wheel loads. Following are the factors which cause the mud pumping :

  (i) Extent of slab deflection (ii) Type of subgrade soil (iii) Amount of free water
- Pumping is noticed just after the rains in cement concrete pavements that are placed on clayey soil subgrade.
- Due to the applications of repeated loads, initial spaces are developed underneath the pavement slabs and water infiltrates into these spaces through joints, cracks and edges of the pavements.
- Since the soil is also of fine grained type, it holds water and forms the soil slurry or soil suspension in water or the *mud*.

PAVEMENT EVALUATION

- Pavement evaluation involves a thorough study of various factors such as (i) subgrade support, (ii) pavement composition and its thickness, (iii) traffic loading and (iv) environmental conditions.
- The various methods of pavement evaluations may be broadly classified into two groups :

  (i) Structural evaluation of pavements (ii) Evaluation of pavement surface condition

Structural Evaluation of Pavements

- Structural evaluation of both flexible and rigid pavement may be carried out by plate bearing test.
- Benkelman Beam measurements are preceded by a rating survey of the road so as to divide it into homogeneous section of approximately similar serviceability.
- A few number of *non-destructive* testing techniques are used for assessing the load carrying capacity of the pavements.

Evaluation of pavement surface condition

- The pavement unevenness may be measured using unevenness indicator, profilograph profilometer or roughometer.
- The *pavement serviceability* concept was introduced at the AASHTO Road Test for comparing relative performance of various test sections during different periods.

- The present serviceability of a pavement is related to a pre-determined scale by a panel of judges sensitive to the wishes of motor vehicle users by actually riding over the pavement.
- The present serviceability rating (PSR) is the mean opinion of the members of the rating panel and this is correlated with the physical measurements such as longitudinal and transverse profile of the pavement, degree of cracking and patching etc. affecting pavement serviceability.

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

Note : Unevenness Index of the surface in cm/km length of road may be called, Bump Integrator.

STRENGTHENING OF EXISTING PAVEMENTS

- For the successful maintenance of pavements, it is essential that they have adequate stability to withstand the design traffic under prevailing climatic and subgrade conditions. If the pavements have to support increased wheel loads and load repetitions, the pavements rapidly undergo the distress and no amount of routine and periodic maintenance can help them.
- Strengthening may be done by providing additional thickness of the pavement of adequate thickness in one or more layers over the existing pavement, which is called ***overlay***.
- If the existing pavements have completely deteriorated, an overlay would not serve the purpose and hence we should remove the existing damaged pavement structure and rebuild it. In partially damaged pavement sections, patch repair works are carried out before constructing the overlay.

Types of Overlay

- The overlay combinations are divided into four categories based on the type of existing pavement and the overlay:

  (i) Flexible overlay over flexible pavements

  (ii) Cement concrete or rigid overlay over flexible pavements

  (iii) Flexible overlay over cement concrete or rigid pavements

  (iv) Cement concrete or rigid overlay over rigid pavements

- The choice of the overlay type depends upon number of factors including total thickness of overlay required, local materials, wheel loads cost etc.

Design of Overlay

The overlay thickness required over a flexible pavement may be determined either by one of the conventional pavement design methods or by a non-destructive testing method like the Benkelman beam deflection method.

Methods of Determining Overlay Requirements

(1) CBR value method

(2) Benkelman Beam deflection method.

CBR Method of Overlay Design

It is a simple method of determining overlay thickness is to find out (i) the CBR value of the subgrade under the pavement, (ii) estimate the thickness needed to cater to future traffic, (iii) measure the existing thickness, and (iv) arrive at the additional requirement by subtraction.

The precautions to be taken in this method are:

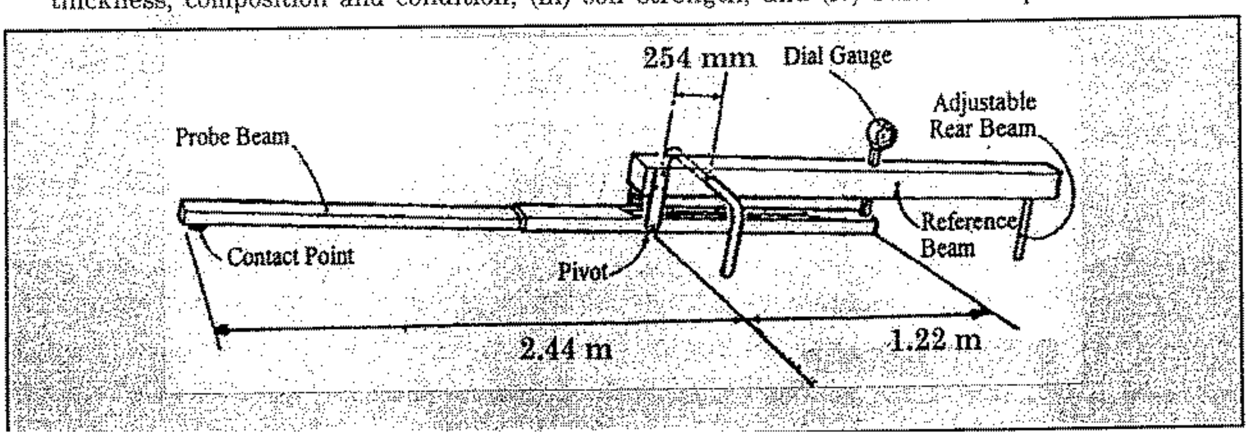
(a) The CBR value of the subgrade under its actual state of compaction and anticipated worst moisture condition is to be found. This can be done by in-situ CBR tests, or CBR tests on undisturbed samples or CBR tests on remoulded samples (compacted to actual density found in the field).

(b) The thickness of various layers needs to be moderated to arrive at a reasonable value of equivalent thickness in terms of some standard material.

Benkelman Beam Deflection Method

Principle of Deflection method or Overlay design:

- A well compacted pavement section or one which has been well conditioned by traffic deforms elastically under each wheel load application (a bowl of deflection is formed under the load) such that when the load moves away, there is an elastic recovery or rebound deflection of the deformed pavement surface.
- The pavement rebounds back to its original shape after the load passes.
- The amount of deflection depends upon a number of factors such as (i) wheel load, (ii) pavement thickness, composition and condition, (iii) soil strength, and (iv) surface temperature



- Larger rebound deflection indicates weaker pavement structure which may require earlier strengthening or higher overlay thickness.
- Deflections can be conveniently measured using the Benkelman beam.

Procedure: The beam is 3.66 m long and is pivoted at a distance of 2.44 m from the contact point. The other end of the beam activates a dial gauge. Thus, if the probe contact point goes down by an amount S, the dial gauge records an upward movement of S/2. The standard method of deflection measurement and overlay thickness design in India is as under.

1. A minimum of 10 points is selected along the outer wheel path for each lane.
2. A single rear axled truck is selected with the rear axle loaded to 81.7 kN (wheel assembly weighing 40.85 kN), the tyre pressure being 0.56 MN/m^2. The probe is inserted in between the two wheels, the wheel position representing the point where the deflection is desired. The contact point is also the same point. The dial gauge reading is noted.
3. The truck is moved forward slowly and the dial gauge readings are taken when the truck is 2.7 m and 9.0 m away from the initial position to give the intermediate and final readings respectively.
4. Pavement temperature is recorded.
5. The intermediate and final readings are subtracted from the initial reading. If the deflections so obtained compare within 0.025 mm, the actual deflection is twice the difference between the initial reading and the final reading.
6. If the deflection obtained from the two positions do not compare within 0.025 mm, twice the difference between the final and initial readings gives the apparent deflection This is known as leg correction.
5. The three deflection dial reading D_o, D_i and D_f form a set of readings at one deflection point under consideration. The truck is moved forward to the next deflection point, the probe of the Benkelman beam inserted and the procedure of noting the set of three deflection observation is repeated. The deflection observations are continued at all the desired points.
7. The mean and standard deviation of the measurements are determined. The characteristic deflection is taken as mean plus two standard deviations.

The allowable deflections are as under :

Design Traffic Intensity (commercial vehicles/day)	Allowable Deflection (mm)
150 – 450	1.50
450 – 1500	1.25
1500 – 4500	1.00

Note: (i) IRC guidelines relate deflection measurements to a standard temperature of 35°C. Corrections are additive when temperatures are less than 35°C and are subtractive when they are more than 35°C. For higher altitudes, measurements are recommended at 20°C with no corrections.

(ii) Deflections are recorded after the rainy season. If they are taken in dry seasons, they should be multiplied by 2 for clayey soils, 1.2 to 1.3 for sandy soils and interpolated for soils in between.

(iii) A loaded truck with rear axle load of 8170 kg is used for the deflection study. The design wheel load is a dual wheel load assembly of gross weight 4085 kg with an inflation pressure of 5.6 kg/cm^2.

Analysis of Data

- The rebound deflection values D_1, D_2, D_3 are determined in mm after applying the leg corrections, if necessary, to the observed values of D_e, D_t and D_1 in each case.

  The mean value of the deflections at n points is given by:

$$D = \frac{\Sigma D}{n} \text{ mm}$$

  The standard deviation of the deflection values is given by:

$$\sigma = \sqrt{\frac{\Sigma(\bar{D} - D)^2}{(n-1)}}$$

  Characteristic deflection D_c is given by:

$$D_c = D + t \times \sigma$$

  When t = 1.0, $D_c = \bar{D} + \sigma$ covers about 84 percent of the cases;

  When t = 2.0, $D_c = \bar{D} + 2\sigma$ covers about 97.7 percent of the cases of deflection values on the pavement section, assuming normal distribution of rebound deflection values.

  The IRC recommends the former case, i.e., $D_c = \bar{D} + \sigma$, whereas in many other countries they adopt the later case for overlay design.

Note: Here the value of 't' is to be chosen depending upon the percentage of the deflection values to be covered in the design.

Overlay Thickness Design

- The overlay thickness required h_0 may be determined after deciding the allowable deflection D_a in the pavement under the design load. According to Ruiz's equation, overlay thickness h_0 in cm is given by :

$$h_0 = \frac{R}{0.434} \log_{10} \frac{D_c}{D_a}, \text{cm}$$

  where, h_0 = thickness of bituminous overlay in cm

  R = deflection reduction factor depending on the overlay material (usual values for bituminous overlays range from 10 to 15, the average values that may be generally taken being 12).

  D_a = allowable deflection which depends upon the pavement type and the desired design life, values ranging from 0.75 to 1.25 mm are generally used in flexible pavements for overlay design.

- The Indian Road Congress suggests the following formula for the design of overlay thickness equivalent to granular material of WBM layer.

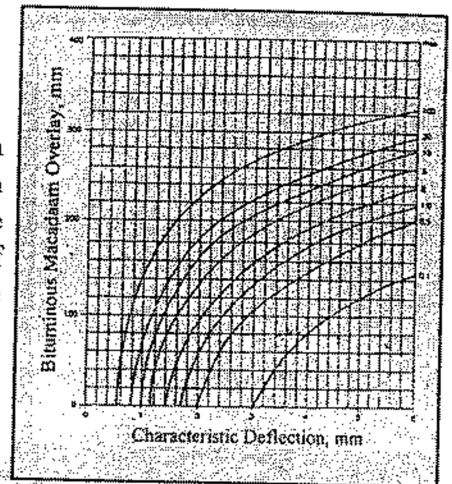
$$h_0 = 550 \log_{10} \frac{D_c}{D_a} \text{ mm}$$

  where, h_0 = thickness of granular or WBM overlay, in mm

  $D_c = (\bar{D} + \sigma)$, after applying the corrections for pavement temperature and subgrade moisture

D_a = 1.00, 1.25 and 1.5 mm, if the projected design traffic A is 1500 to 4500, 450 to 1500 and 150 to 450 respectively.

Here, A = design traffic = $P[1 + r]^{(n+10)}$



- When bituminous concrete or Bituminous Macadam with bituminous surface course is provided as the overlay, an equivalency factor of 2.0 is suggested by the IRC to decide the actual overlay thickness required. Thus, the thickness of bituminous concrete overlay in mm will be $h_0/2$ when the value of h_0 is determined from Equation.

Example 1

Benkelman beam test was carried out on a stretch of flexible pavement of a National Highway, as per IRC guidelines. The mean value of deflections and standard deviations computed are 1.57 mm and 0.12 mm respectively. The other data are:

(i) Temperature during the test = 25°C

(ii) Correction factor for subgrade moisture content = 1.2

(iii) Initial commercial traffic in the year of completion of construction of overlay: 1200 veh/day

(iv) Annual growth rate of commercial vehicles = 7.5%

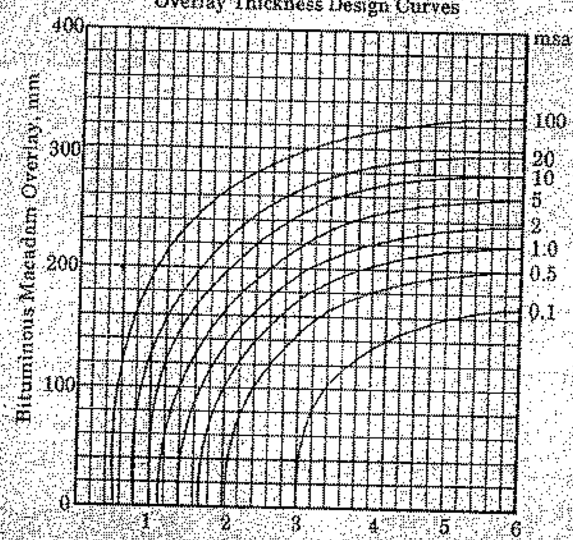
(v) Design Life = 10 years

(vi) Vehicle damage factor = 3.5

Take 1 cm of bituminous macadam equal to 0.7 cm of DBM/AC/SDC.

Determine the thickness of asphaltic concrete overlay using the figure below.

Overlay Thickness Design Curves



Sol. Data Given : Test temperature = 25°C

Correction Factor for subgrade moisture content = 1.2

Initial Commercial Traffic, P = 120 Vehicle/day

Annual Growth Rate, r = 7.5% ; Design life = 10 year

Mean Deflection, $\bar{D}$ = 1.57 mm ; Standard Deviation, σ = 6 = 0.12 mm

$$\text{Design Commercial Traffic} = P\left(1+\frac{r}{100}\right)^n = 1200\left(1+\frac{75}{100}\right)^{10} = 2474 \text{ Vehicle / day}$$

$$\text{Characteristic Deflection } D_c = (\bar{D}+\sigma) = 1.57 + 0.12 = 1.69 \text{ mm}$$

IRC has suggested for a standard pavement temperature of 35°C and a correction factor of 0.0065 mm°C to be applied for the variation from the standard pavement temperature. The correction will be –ve when the pavement temperature is above 35°C and positive when it is lower.

$\therefore$ Deflection after temperature correction = $1.69 + 0.0065(35-25) = 1.755$ mm

As per IRC, $$h_0 = 550\log_{10}\left(\frac{D_C'}{D_a}\right)$$

where, h_0 = Thickness of groundwater or WBM over lay in mm

D_C = Characteristic deflection after correction

D_a = Allowable deflection which is equal to 1 mm for a design traffic between 1500 and 4500 Vehicle/day

$$\therefore \quad h_0 = 550\log_{10}\left(\frac{2.11}{1}\right) = 178.36 \text{ mm}$$

When bituminous concrete or bituminous macadam with bituminous surface course is provided on the overlay, an equivalency factor 2 is suggested by IRC to decide the actual overlay thickness required.

$$\text{Thickness of bituminous macadam AC} = \frac{178.36}{2} = 89.18 \text{ mm}$$

$$\text{Allow 1 cm of bituminous macadam} = 0.7 \text{ of AC} = 0.7 \times 89.18 = 62.42 \text{ mm}$$

Example 2

Determine the thickness for an existing pavement when the mean deflection observed was 1.50 mm, the standard deviation was 0.30 mm, the temperature of observation was 35° C. Traffic at the end of 10 years is 3000 commercial vehicle per day.

Sol. No temperature corrections are needed in this case as the readings were taken at 35°C

$\Delta_0 = 1.50 + 2 \times 0.30 = 2.10$ mm ; $\Delta = 1.00$

$$h = 550\times\log_{10}\frac{2.10}{1.00} = 550 \times \log_{10} 2.10 = 550 \times 0.322 = 177 \text{ mm (granular).}$$

$$\text{With bitumen bound layers overlay thickness} = \frac{177}{2} = 90\text{mm.}$$

Provide 50 mm bituminous macadam and 40 mm asphaltic concrete.

Example 3

The mean and standard deviation of deflection measurements are 1.00 mm and 0.25 mm respectively. The temperature at the time of measurements was 40°C. The measurements were taken in the dry season. The soil is clayey. The design traffic at the end of 10 years is 2000 commercial vehicles per day. Design the overlay.

Sol. Allowable deflection = 1.0 mm

Characteristic deflection = 1.00 + 0.25 × 2 = 1.50 m

Temperature correction = (40 – 35) 0.0065 = 0.03

Deflection after temperature correction = 1.50 – 0.03 = 1.47

Defl. after correction for seasonal variation = 1.47 × 2.0 = 2.94 mm

$$h = 550 \log_{10} (2.94/1.00) = 258 \text{ mm}$$

Provide 100 mm water-bound macadam layer to be overlaid with bitumen-bound layers.

Thickness of bitumen-bound layers required = (258 – 100)/2 = 79 mm

Provide 50 mm bitumen macadam and 40 mm asphaltic concrete.

OBJECTIVE QUESTIONS

1. **Assertion (A):** Bituminous roads disintegrate even with light traffic, but such road failures are not due to any wrong use of surface treatment.

   Reason (R): Improper preparation of the subgrade and the foundation are responsible for this disintegration.

2. Match **List-I** (Pavement deficiency) with **List-II** (Explanation) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Bird baths	**1.** A step-sided, bowl shaped cavity caused by loss of surfacing as well as base course erosion
B. Pot holes	**2.** Deformation which may be caused by localized or variable subgrade failure
C. Ravelling	**3.** Irregular deformations which may be the result of differential settlement
D. Subsidence	**4.** Removal of larger surface aggregates leaving craters
	5. Abrupt lowering of the road surface due to poor drainage

Codes:

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

3. Consider the following:
   1. LL of soil
   2. PL of soil
   3. SL of soil
   4. Annual average rainfall
   5. Temperature of the soil

   As per the latest IRC guidelines, the set of essential data required to determine moisture correction factor of clayey subgrade soil in Benkelman beam study would include

   (a) 1, 2 and 4 (b) 1, 2, 3 and 4
   (c) 2, 3 and 4 (d) 4 and 5

4. For carrying out bituminous patch work during the rainy season, the most suitable binder is

   (a) road tar (b) hot bitumen
   (c) cutback bitumen (d) bituminous emulsion

5. The corrected characteristic rebound deflection on a pavement, using Benkelman beam study is 2 mm. The equivalent granular overlay thickness required for an allowable deflection of 1 mm as per original IRC guidelines is

   (a) 33 mm (b) 66 mm
   (c) 133 mm (d) 166 mm

6. Which one of the following is useful in functional evaluation of pavement?

   (a) PCU (b) PSI
   (c) PIEV (d) Benkelman beam

7. Match **List-I** (Distress) with **List-II** (Meaning) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Settlement	**1.** A localized, rather abrupt lowering of the road surface
B. Subsidence	**2.** A general lowering of the road surface
C. Depression	**3.** Irregular deformation of the road
D. Distortion	**4.** Dished localized deformations

Codes:

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

8. On a National Highway pavement section mean value of Benkelman beam deflection was obtained as 1.45 mm with standard deviation σ of 0.107 mm. Pavement temperature measured was 37°C and moisture correction was obtained as 1.6. What is the value of characteristic deflection considering the value of 2σ?

(a) 2.72 mm (b) 2.69 mm
(c) 2.63 mm (d) 1.94 mm

9. Which one of the following defects indicates progressive disintegration of bituminous premix carpet surfacing by loss of aggregates?

(a) Pot holes (b) Ravelling
(c) Edge breaking (d) Rutting

10. Match **List-I** (Road drainage system) with **List-II** (Location) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Vertical sand drains	**1.** Urban road drainage
B. Causeways and culverts	**2.** Hill roads
C. Scuppers and catch water drains	**3.** High embankments in soft soils
D. Inlets and gratings	**4.** Cross drainage in road alignments

Codes:

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

11. A road surface is corrected by spreading a layer of dry sand in a thickness varying from 5 mm to 10 mm and rolling the surface by heavy rollers. Which one of the following maintenance works does it apply to?

(a) Repair of ruts and patches (b) Repairing of blow ups
(c) Repair of bleeding surface (d) Sealing of joints and cracks

12. On a National Highway, GSB may be used as
    (a) sub-base and drainage layer
    (b) drainage layer and prime coat
    (c) wearing course and drainage layer
    (d) sub-base and tack coat

13. The phenomenon of reflection cracking is observed in which one of the following?
    (a) Bituminous overlays provided over cracked cement concrete or bituminous pavements
    (b) Cement concrete overlays provided over cracked bituminous pavements
    (c) Bituminous overlays constructed in cold weather
    (d) Cement concrete overlays provided over cement concrete pavements

14. Consider the following statements:
    1. Mastic asphalt has self-healing property
    2. Mastic asphalt contains more percentage of bitumen.
    3. Synthetic geotextile is used to make mastic asphalt.
    4. Mastic asphalt is used in heavy duty areas.

    Which of these statements are correct?
    (a) 1, 2 and 3 (b) 3 and 4
    (c) 1, 2 and 4 (d) 2, 3 and 4

15. Which type of pavement overlay needs to be designed by Benkelman beam deflection method?
    (a) Rigid overlay on rigid pavement
    (b) Flexible overlay on rigid pavement
    (c) Rigid overlay on flexible pavement
    (d) Flexible overlay on flexible pavement

ANSWERS

1.	(a)	2.	(d)	3.	(a)	4.	(d)	5.	(d)	6.	(b)	7.	(a)
8.	(c)	9.	(a)	10.	(c)	11.	(b)	12.	(a)	13.	(a)	14.	(c)
15.	(d)												

CHAPTER

8

Hill Road

INTRODUCTION

- A hill road is one which passes through a terrain with a cross slope of 25 percent or more.
- Geometric standards as of plains cannot be adopted in hills. Massive and costly protective works are required at many places in the hill alignment resulting in heavy expenditure.
- Normally hill roads are also classified as N.H., S.H., M.D.R., O.D.R and V.R. as in level terrain.
- The Border Roads Organisation has classified hill road as follows.
  (i) National Highways
  (ii) Class 9 (6 m wide for 3-tonnes vehicles)
  (iii) ~~Class 5 (4.9 m wide for 1-tone vehicles)~~
  (iv) Class 3 (2.45 to 3.65 m wide for jeeps)

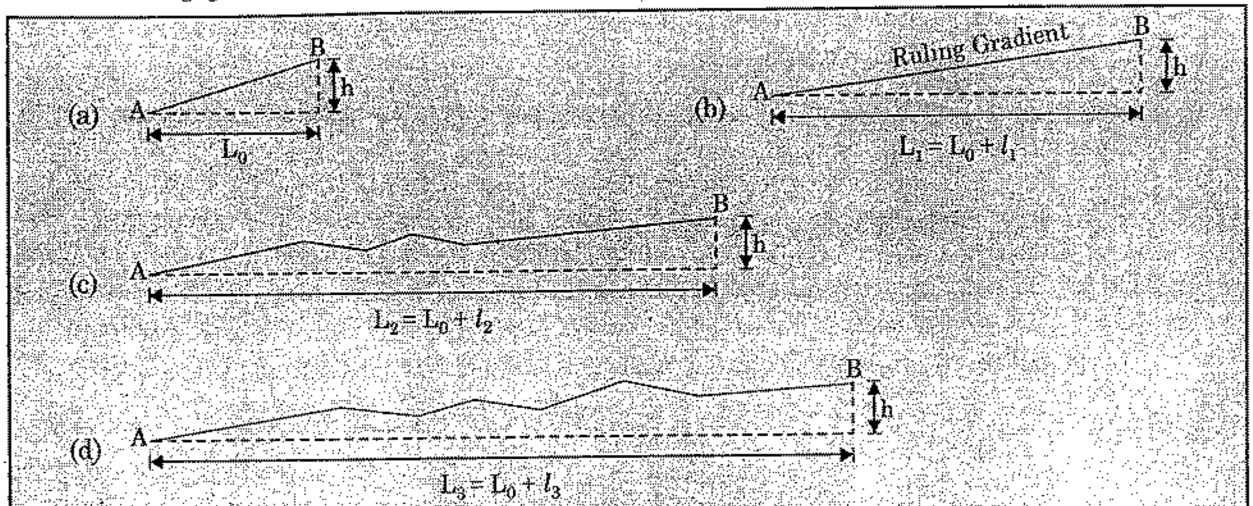
ALIGNMENT OF HILL ROADS

- The hill road alignment should link up the obligatory and control points fitting well in the landscape and satisfying the geometric requirements.
- The best alignment for a hill road is one wherein the total sum of the ascends and descends between extreme points is the least.
- It is permissible to increase the length as much as 50 times the height saved by a detour.
- Aerial survery and photogrammetric applications are most useful for the survey of the hills.
- Some particulars of special significance are discussed below:

Resisting Length

- The resisting length of a road is its effective length taking into consideration the total work done against the resistances.
- In hill stations the difference in elevation 'h' is likely to be high when compared to the shortest distance L_0 resulting in a gradient steeper than the ruling gradient as in Figure (a).
- $L_1 = (L_0 + l_1)$ so as to have the desired ruling gradient Fig. (b).
- In actual practice it is not possible to strictly follow such uniform rate of gradient.
- Longitudinal sections of two alternate alignments with length $L_2 = (L_0 + l_2)$ and $L_3 = (L_0 + l_3)$ respectively. These two alignments also have ineffective rises and falls, the sum of which are h_2 and h_3 respectively.

- The sum of ineffective rise and fall are obtained by finding the total rise and total fall in excess of floating gradient then subtracting the actual difference



Longitudinal Section of alternate routes

- The total work done in Fig. (a) in moving a load W from station A to B along the shortest length L_0 up to height,

$$h = WfL_0 + Wh = Wf(L_0 + h/f) = WfL_r$$

where, L_r = resisting length = $L_0 + h/f$; f = coefficient of frictional resistance.

In case (b) the resisting length $L_{r1} = L_1 + h/f$

Similarly in alignments (c) and (d) the values of resisting lengths are given by

$$L_{r_2} = L_2 + \frac{h+h_2}{f} = L_0 + l_2 + \frac{h+h_2}{f}$$

and

$$L_{r_3} = L_3 + \frac{h+h_3}{f} = L_3 + l_3 + \frac{h+h_3}{f}$$

- In these equations l_2 and l_3 are the additional distance to be traversed along the alternative routes, h_2 and h_3 are the total ineffective rise and fall along the routes.

Trace Cut for Hair Pin Bends

- Trace cut is a narrow track, 0.6 to 1.0 metre wide, prepared along the alignment of the hill road to enable access for inspection during location of the route.
- In hill roads trace cut is done at a ruling gradient of 1 in 25.
- The ruling gradient is changed to almost level at the hair-pin bends sharp cover to ensure that finished gradient is correct. Hence detailed survey is necessary at each sharp curve, particularly at hair-pin bends.

Geological Considerations

- Degree of stability of slope depends on type of rock, inclination or dip of strata and presence of ground water. Dip of the strata should be as small as possible or alternatively, be inclineded away from it.

Alignment Survey

The alignment of hill road is fixed in three stages:

(i) Reconnaissance (ii) Trace cut (iii) Detailed surveys

GEOMETRICS OF HILL ROAD

- The geometric standards for gradient, superelevation, and radius of curves etc. on hill roads are different from those in the plains.
- The main reasons for the difference are the topography and other problems in alignment of hill raods.

Width of Pavement, Formation and Land

- At stretch on hard rock, the shoulders may be reduced by 0.4 m on either side on two-lane roads and by 0.2 m in other cases. The minimum set back for building line beyond the right-of-way should be 5m in normal cases and 3 m in exceptional circumstances.

Camber or Cross-Fall

- Steeper cross slope or camber is adopted for hill roads and recommended values are given in Table.

Width of Pavement, Formation and Land

Highway classification	Pavement width m	Roadway width, m (excluding side drains and parapets)	Right-of-way width, m	
			Normal	Exceptional
NH & SH				
Two-lane	7.00	8.80	24	18
single-lane	3.75	6.25	24	18
MDR	3.75	4.75	18	15
ODR	3.75	4.75	15	12
VR	3.00	4.00	9	9

Recommended values of camber

Type of Surface	Camber, percent
Subgrades, earth roads and shoulder	3.0 to 4.0
Gravel and W.B.M. surface	2.5 to 3.0
Bituminous surfacings	2.5
High type bituminous surface & CC	2.0

- When the road has longitudinal gradients greater than 1 in 20, flatter camber may be provided.

Sight Distance

- The stopping sight distance is calculated from the relation:

$$SSD = 0.278\, Vt + \frac{V^2}{254f}$$

where, V = design speed, kmph ; t = reaction time, taken as 3 seconds

f = coefficient of friction, assumed as 0.4.

- Safe stopping sight distance for various speeds given by IRC are given below:

Speed	20	25	30	40	50
SSD, m	20	30	35	50	70

- The overtaking sight distance is calculated from the relation:

$$OSD = 0.556\ V_b + 2s + 0.278\ TV_b + 0.278\ VT$$

where,

V = speed of overtaking vehicle, kmph

V_b = speed of overtaken vehicle = (V – 16) kmph

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

T = overtaking time = $\sqrt{\dfrac{14.4}{A}}$, secs

A = acceleration in kmph/sec. taken as 4.72, 4.45 and 4.0 for speeds of 30, 40 and 50 kmph respectively.

Minimum overtaking sight distance specified are:

Speed, kmph	30	40	50
OSD, m	90	145	210

Superelevation

- The superelevation to be provided at horizontal curves of hill roads is calculated from the formula:

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

- IRC specifies that the superelevation should not exceed 7 percent in sections of hill roads which get snow bound and 10 percent in other places.

Radius of Horizontal Curve

- The minimum radius of horizontal curves in hill roads, R (min) is calculated from the formula:

$$R\ (\min) = \frac{0.008V^2}{e+f}$$

- The lateral friction factor f is taken as 0.15. The minimum radii recommended for various classes of hill roads are given in Table.
- Reverse curves are designed to have a minimum radius of 30m for the compound curves and a straight distance of 9 m between their transitional ends.

Minimum Radial of Curves in Hill Roads

Category of Roads	Minimum radius, metre			
	Mountainous terrain		Steep terrain	
	not snow bound	snow bound	not snow bound	snow bound
N.H. & S.H.	50	60	30	33
M.D.R.	30	33	14	15
O.R.D.	20	23	14	15
V.R.	14	15	14	15

Widening at Curves

- Extra width of carriageway W at horizontal curve is calculated from the relation:

$$W_e = \frac{18n}{R} + \frac{0.1V}{VR}$$

where n is the number lf lanes.

- The recommended values of extra widening on single and two-lane pavements at curves are given below for various speeds.

Radius of curve, m	14 to 20	20 to 30	30 to 60	60 to 150	above 150
Extra width of single-lane roads, m	1.5	1.2	0.9	0.6	Nil
Radius of curve, m	30 to 40	40 to 60	60 to 100	100 to 150	above 150
Extra width of two-lane roads, m	1.5	1.2	0.9	0.6	Nil

Setback Distance

- As it is not practicable to provide visibility corresponding to overtaking sight distance all along the hill road, the alignment is made so as to provide atleast the safe stopping sight distance.

Transition Curves

- The length of transition curves is to be calculated from the formula :

$$L_s = \frac{0.0215V^2}{CR}$$

where, $C = \frac{80}{V+75}$ (maximum values of 0.76 for speeds less than 30 kmph)

L_s = length of transition, metre ; R = radius, metre ; V = design speed, kmph

- The minimum lengths of transition recommended by I.R.C. are 10 m for design speed up to 40 kmph and 20 m for design speed 40 to 50 kmph.

Gradients

- The ruling and limiting gradients in mountainous terrain and in steep terrain over 3000 m height above mean sea level are 5 and 6 percent respectively.
- The ruling and limiting gradients in steep terrain upto 3000 m height above MSL are 6 and 7 percent.

HAIR PIN BENDS

- In aligning a hill road, it becomes sometimes necessary to attain height at a particular location, without substantial covering of horizontal distance. In such cases, it is customary to introduce a series of hair-pin bends.

1.	Minimum design speed	20 km/hr
2.	Minimum width at apex	
	(i)National Highways and	
	State Highways	
	Double lane	11.5 m
	Single lane	9.0 m
	(ii)Major District Roads	
	Other District Roads	7.5 m
	(iii)Village Roads	6.5 m
3.	Minimum radius of inner curve	14.0 m
4.	Minimum length of transition	15.0 m
5.	Gradient	
	Maximum	1 in 40
	Minimum	1 in 200
6.	Superelevation	1 in 10

- The distance between two consecutive hair-pin bends should be a minimum of 60 m.

MISCELLANEOUS STRUCTURES IN HILL ROADS

- A variety of structures is met with in hill road design. Of these, the following are common:
  1. Retaining walls
  2. Parapet walls
  3. Breast walls
  4. Check walls
  5. Catch-pits
  6. Snow-chutes.

Retaining walls

- Retaining walls are needed to retain the fill portion of the highway cross-section. Due to the warps of the hill faces, it often becomes necessary to take the road partly in filling and partly in cutting, fully in cutting or fully in filling.
- In such circumstances, retaining walls become necessary.

Parapet walls

- Parapet walls are needed to give protection, psychologically and physically, to the motorists while travelling on roads with steep valley slopes.

Breast walls

- Breast walls are constructed to buttress the uphill slopes of the road cross-section. They should be stout enough to withstand the earth pressure of the soil behind, along with the surcharge, caused by the slope.

Cbeckwalls

- Checkwalls are small retaining structures constructed in series on a sloping hill face to check the slides and to generally add to the overall stability of the hill-face.
- The top width is generally kept 0.6 m and for a height of 1.5m, a bottom width of 1.0 m is provided.
- The top and bottom 0.15 m are set in cement mortar, whereas the remaining height is made in dry masonry.

Gabion walls

- Retaining walls, breast walls and checkwalls can be constructed with dry stone masonry encased in wire mesh. Such a construction is called gabion wall and is popular in several developing countries.
- The advantage is that the gabion walls can adjust themselves easily, being flexible in nature, to the settlement or disturbances that normally take place and hence do not get damaged.
- Gabion walls are also used as toe-protection walls where the road runs parallel to a stream

LANDSLIDES

- Landslides are a common problem encountered in hill roads, and more so in the Himalayan hill ranges in India. This is because these hill ranges are very young geologically and are subjected to adjustments and settlements.

Factors contributing to land-slides

- The factors contributing to land-slides can be discussed under the following groupings:
  1. Factors contributing to increases shear stress
  2. Factors contributing to reduced shear strength.

Correction of slides

- Land-slides can be corrected in a number of ways. The efficacy of each treatment depends upon the particular conditions prevailing at site.
- A combination of some of the well-known treatments work in most situations. Somes of the remedies are
  1. Improvement in the surface drainage by catchwater drains, side drains and cross-drainage structures
  2. Improvement in the sub-surface drainage by burried French drains.

3. Afforestaion.
4. Prevention of grazing of animals by fencing.
5. Grouting, rock-bolting, dowelling.
6. Construction of check-walls, breast-walls and toe walls.
7. Asphalt-mulch treatment of the slopes and growth of vegetation.
8. Jute-netting and wire-netting.
9. Chemical treatment.

Control of snow drift through highway location

- It is possible to control the drift of snow by proper design of the highway. Some of the points that deserve consideration in this regard are:
  1. When crossing valleys, it is better to choose locations passing through tall forests so that the trees can shield the road from winds and trap the snow.
  2. In rolling country, the leeward side of the natural slopes should be avoided and the road should be located on the windward side or at the crest, if possible.
  3. The finished grade-line should be higher than the height of the vegetation and the top of the snow deposition on the adjoining ground.
  4. The highway cross-section should have as little obstruction as possible. Even highway appurtenances like curbs, guard-rails, traffic signs etc. can cause accumulation of snow.
  5. In special problematic areas, snow fences, located perpendicular to the direction of the wind on the upwind side, are erected. The distance of such fences from the highway should be 10–15 times the height of the fence plus 5 m. Rows of fences can be provided instead of a single row.

Snow clearance

- Manual methods of snow clearance are slow and cannot cope up with the needs of high altitude snow removal. Machinery is the only answer. The machinery employed can be of the following types:
  1. One-way blade plough
  2. Two-way blade plough
  3. Rotary ploughs.

Salt application

- Salt application is commonly used for treatment icy conditions. The salt brings about melting of ice at temperatures below the normal freezing point i.e. 0°C. Sodium chloride and calcium chloride are commonly used. Normally 90—180 kg of salt is needed per km length of the two-lane road.

OBJECTIVE QUESTIONS

1. In case of hill roads, which one of the following is correct?
   (a) Resisting length should be kept as low as possible
   (b) Resisting length should be kept as large as possible
   (c) There is no relevance for resisting length
   (d) Resisting length should be equal to stopping sight distance
2. Which one of the following items of hill road construction does not help in the prevention of landslides in the monsoon season?
   (a) Retaining walls
   (b) Catch water drains
   (c) Breast walls
   (d) Hair-pin bends
3. While designing a hill road with a ruling gradient of 6%, if a sharp horizontal curve of 50 m radius is encountered, the compensated gradient at the curve as per the Indian Roads Congress specifications should be
   (a) 4.4%
   (b) 4.75%
   (c) 5.0%
   (d) 5.25%
4. Match **List-I** (Type of wall) with **List-II** (Feature) and select the correct answer using the code given below the lists:

List-I	List-II
A. Parapet wall	**1.** Constructed with dry stone masonry encased in wire mesh
B. Check wall	**2.** To add the overall stability to the hill face
C. Breast wall	**3.** To buttress the uphill slopes of the road cross-section
D. Gabion wall	**4.** To give protection to the motorists

Codes:

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

ANSWERS

1. **(a)** **2.** **(d)** **3.** **(a)** **4.** **(c)**