UNIT-I

Objective:

To familiarize with different concepts in all modes of Transportation.

Syllabus: Types of pavements; Functions and Requirements of different components of pavements. Railway Engineering-Permanent way components, Cross section of Permanent way. Airport Engineering: Basic elements of airport-Runway, Taxiway, Apron, Terminal Building and Hanger, Aircraft and its characteristics. Harbour Engineering- Classification of ports and harbours, requirements of a good port.

INTRODUCTION

1.PAVEMENT: Pavement can be defined as "The combination of several layers, constructed over prepared soil, in order to resist the Wheel Loads of traffic and transmit them safely to the foundation soil". The Pavement structure is strong, stable and durable the entire design period to serve traffic needs.

1.1 FUNCTIONS & REQUIRMENTS OF PAVEMENT:

- It should be strong and smooth surface to resist traffic loads.
- Distribute the loads safely to the foundation soil through the intermediate layers
- Carry the repeated loads without developing excessive deformations.

In order to fulfil these **Functions**, the **Requirements** of pavements are:

1. It should be structurally strong to withstand the stresses imposed by traffic
2. Thickness of load should be adequate to distribute the loads.
3. Hard wearing surfaces are provided to resist abrasion caused by vehicles
4. Provide enough friction for tractive effort and to prevent skidding.
5. It should not affected by water.
6. Its Initial cost and Maintenance cost should be minimum

2.TYPES OF PAVEMENTS

Based on the structural behaviour, Road pavements are generally classified into Four categories, namely:

1) Flexible Pavements.
2) Rigid Pavements.
3) Semi-Rigid Pavements and Composite Pavements.
4) Interlocking cement concrete block Pavement.

2.1.FLEXIBLE PAVEMENTS:

Flexible Pavement can be defined as "Pavement layer comprising of a mixture of Aggregates & Bitumen, heated and mixed properly, then laid and compacted on a bed of granular layer". It is a Multi-layered system with low flexural strength. The external loads are largely transmitted to the subgrade through the layers. The load distribution capacity of each layers depends upon the nature of the materials and mix design aspects.

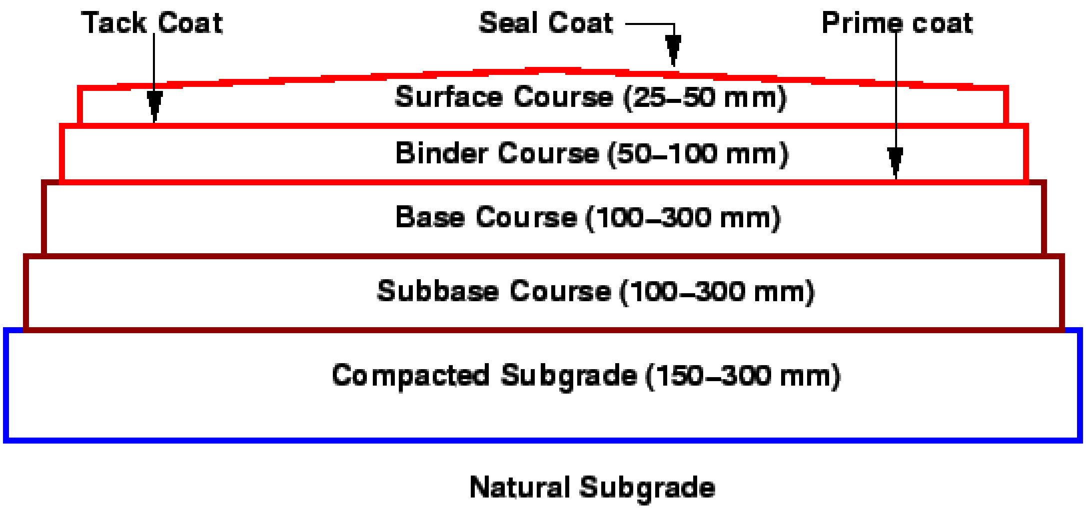


Fig 2.1.1 Cross section of Pavement.

The Surface Course, which is direct contact with traffic loads has to be Strongest, while the layers below can be of relatively lower strength. Surface course consists of a mix with a binder material like bitumen and aggregates. The base and sub-base courses consist of granular materials like crushed stone aggregate, gravel and aggregate-soil mixes. Material which is used in base and sub-base courses are slightly different in specifications.

2.2.RIGID PAVEMENTS:

The Rigid pavements are generally made of Portland Cement Concrete (CC) and also called as "CC Pavements". It is provided with transverse and longitudinal joints. It prevents ejection of soil slurry through joints and cracks. These pavements have high flexural strength than flexible pavements. Flexural strength allows pavement to bridge over minor irregularities in subgrade or other courses which it rests. The primary difference between rigid pavement and flexible pavement is structural behaviour. Flexural stress is high in pavement slab not only due to wheel load, but also warping caused by changes in temperature in summer and winter seasons and during the day and night.

2.3.SEMI-RIGID PAVEMENTS AND COMPOSITE PAVEMENTS.

Bonded materials like POZZOLANIC Concrete (lime fly-ash aggregate mix), lean cement concrete or soil-cement are used in sub base course of pavement layer. Some chemicals are used for soil stabilization to form a semi-rigid layer. These bonded materials have significant flexural strength than flexible pavements and also less flexural strength comparatively with CC pavements. Semi-rigid materials are used in sub-base and base course layer of pavements, they are called as Semi-rigid pavements. The pavements consisting of both flexible pavement layers and one or more semi-rigid pavement layers are called as composite pavements.These pavements have low resistance to impact and abrasion, then they are not used in surface course. There is a need to provide a bituminous surface course or granular base and bituminous surface course over the semi-rigid layer.

2.4.INTERLOCKING CEMENT CONCRETE BLOCK PAVEMENT. (ICBP)

It consists of a layer of cement concrete paver blocks (CPB) of specified strength, size and shape properly laid over well compacted soil subgrade, sub-base and base course layers. The gap between paver blocks are filled with joint filling sand and vibrated to provide adequate interlocking between blocks. ICBP could advantageously laid as surface course over well designed flexible pavement layer system with appropriate drainage/GSB and base course layers. This type of pavement is constructed in water-logged areas, at road intersections and areas where there is a chance of dripping of fuel.

2.5.COMPARISION OF FLEXIBLE & RIGID PAVEMENTS:

S.No	Flexible Pavements	Rigid Pavements
1	It is a multi-layer structure with materials of highest quality near the surface	It consists mainly of cement concrete slab with flexural strength, which can also serve as a wearing course
2	Flexible pavement design procedures are mainly empirical	Rigid pavement design procedures are more precise, as flexural behaviour of concrete is well understood.
3	It reflects the deformations of subgrade, sub-base and base courses on the surface	It can bridge over local weak spots & deformations without reflecting them on surface
4	These pavements depend on subgrade strength for their performance, besides base and sub-base for safe transmission of loads	These pavements depend on flexural strength of concrete slab for safe transmission of traffic loads
5	Transmission of stresses to the subgrade is through the component courses	Distribution of loads to wider area of subgrade depends on the rigidity, high elastic modulus and flexural strength of pavement
6	Stability depends upon interlocking and friction between aggregates and soil cohesion	Stability is derived by the structure strength of the pavement by its slab action
7	The life of flexible pavement ranges from 10 to 20 years	The life of rigid pavement is about 40 years
8	Initial cost is less	Initial cost is more
9	Maintenance cost is more	Maintenance cost is less
10	Riding quality is not good for thin bituminous layers	Riding quality is good

3.FUNCTIONS & REQUIRMENTS OF DIFFERENT COMPONENTS OF PAVEMENTS

3.1.FLEXIBLE PAVEMENT:

The main components of flexible pavements are

a) Sub-Grade
b) Sub-Base and Drainage layer
c) Base course
d) Surface Course/ Bituminous Binder

a)Sub-Grade:

Sub-Grade is compacted natural soil immediately below the pavement layers, and it is act as foundation for highway. Top surface of the subgrade is called the FORMATION LEVEL. Depending upon the alignment and nature of terrain, Roadway is constructed over the embankment or cutting or nearly at the ground level. Formation level has to be properly decided to suit these conditions. The minimum thickness of compacted subgrade is 500 mm on National and State Highways, Major Arterial roads. For Rural roads which carry low volume of traffic, thickness is provided at 300 mm in India. It is also necessary to keep the subgrade and other pavements layers well drained to retain maximum possible strength.

Several tests are conducted to evaluate the strength properties of subgrade of soil. The strength tests are commonly adopted for the evaluation of soil subgrade are:

California Bearing Ratio (CBR) test
Dynamic cone Penetrometer (DCP) test
Triaxial compression or direct shear test
Plate bearing test

CBR test is carried out in the laboratory on soil specimens compacted to desired density and soaked in water. This test also carried out to evaluate the strength of other flexible pavement component materials.

DCP test is used to evaluate the strength of characteristics of subgrade soil in-situ and essentially a field test. However it is necessary to know the limitations of the test before interpreting the test results.

Triaxial compression test is considered to asses the strength characteristics of soil such as cohesion and friction coefficient of soil. However not carried out for sub-grade soil.

b)Sub-Base and Drainage layer

This is immediately below the base course and immediately above the Sub-grade layer, and provide additional help to the courses above on it in distributing loads. Sub-base course has to serve as an effective drainage layer of pavements and also sustain lower magnitude of compressive stresses than the base course. Crushed stone aggregates with good permeability are used in this layer and serves as drainage layer. Coarse aggregates with low percentage of fines (less than 5% finer than 0.075) will serve as a good drainage layer. This layer is covering with full width of formation between the longitudinal drains. The part of the rain water which may enter into the pavement layers through the shoulders or the pavement surface will get drained out quickly into the longitudinal or road side drains. Thus it is possible to retain the subgrade and other pavement layers in relatively dry condition.

c)Base course:

The base course is considered as most important component of flexible pavement layer. This course comes immediately below the surface course and immediately above the sub-base course. The main function of this layer is to distribute the stresses transmitted through the surface course evenly onto the layers below. It consists of granular or bituminous material and act as structural part of the pavement. As per MORTH (Ministry of Road Transport and Highways), the aggregates used in base course should have aggregate value impact value (less than 30%) and Los Angels abrasion value (less than 40%).

d)Surface Course/ Bituminous Binder:

This is the Top most layer and its function is to provide a smooth, strong, abrasion-resistant and reasonably impervious course. Since it is directly contact with the vehicle tyres, it has to resist the imposed wheel loads and transmit them safely to the layer below. The material may be granular, bituminous or cement concrete depending upon the nature of construction. With a good surfacing and an effective drainage layer, it is possible to keep soil subgrade in dry condition. Bituminous surface courses of different types and specifications are used in india. Thin bituminous layers such as Surface dressing, 20 mm thick Pre-mixed bituminous carpet with seal coat and 20 mm thick mixed seal surface are commonly adopted in the wearing course of roads.

3.2RIGID PAVEMENTS:

The components of a typical rigid pavement or cement concrete (CC) pavement structure from bottom towards top consists of

a. Subgrade
b. Granular Sub-base Course and drainage layer
c. Base Course
d. CC/PQC pavement slab

a)SUBGRADE:

The subgrade is the lowest layer of the components of CC pavement which ultimately supports all other component layers and traffic loads. It consists of natural or selected soil with required specifications and well compacted in layers to specified density and thickness. The subgrade yields due to improper compaction, then different types of failures start developing in rigid pavements also. The strength test commonly adopted for evaluation of subgrade for rigid pavement design is “Plate bearing test”.

b)GRANULAR SUB-BASE COURSE AND DRAINAGE LAYER(GSB)

Granular sub-base course has to serve as an effective drainage layer of rigid pavement to prevent moisture content in the subgrade soil. GSB course comes immediate above on subgrade and immediate below of base course. Crushed stone aggregates are preferred in the granular sub-base course with high permeability and serves as an effective drainage layer. Coarse graded aggregates with low percentage of fines (less than 5% finer than 0.075 mm size) will serve as a good drainage layer. An effective drainage layer under cc pavement has following benefits:

- Increase service life and improved performance of cc pavements.
- Prevention of early failures of rigid pavement due to “pumping and blowing”.
- Protection of subgrade against frost action.

c)BASE COURSE:

Base course is immediate layer below CC/PQC pavement slab and immediate layer above granular sub-base course. Base course is generally provided under the CC pavement slab in low and moderate traffic roads. Roads carrying heavy to very heavy traffic loads, high quality base course materials such as lean cement concrete or “dry lean concrete”(DLC) is preferred in base course as they are designed for a life of 30 years or more.

d)PQC PAVEMENT SLAB:

As per IRC(Indian Road Congress) M-40 cement concrete mix with minimum flexural strength of 45 kg/cm^2 is recommended for use in CC pavements of highways with heavy to very heavy traffic loads. CC pavement slab is expected to withstand flexural stress caused by heavy traffic loads and warping effects due to temperature. Steel reinforcement is provided at mid depth of CC pavement slab to avoid stresses and warping effects in the slabs.

4.PERMANENT WAY COMPONENTS:

Permanent way is the generic term for the track (rails, sleepers and ballast) on which railway trains run. Although the configuration of the track today would be recognized by engineers of the 19th century, it has developed significantly over the years as technological improvements became available, and as the demands of train operation increased. The cross section of permanent way is shown in Fig

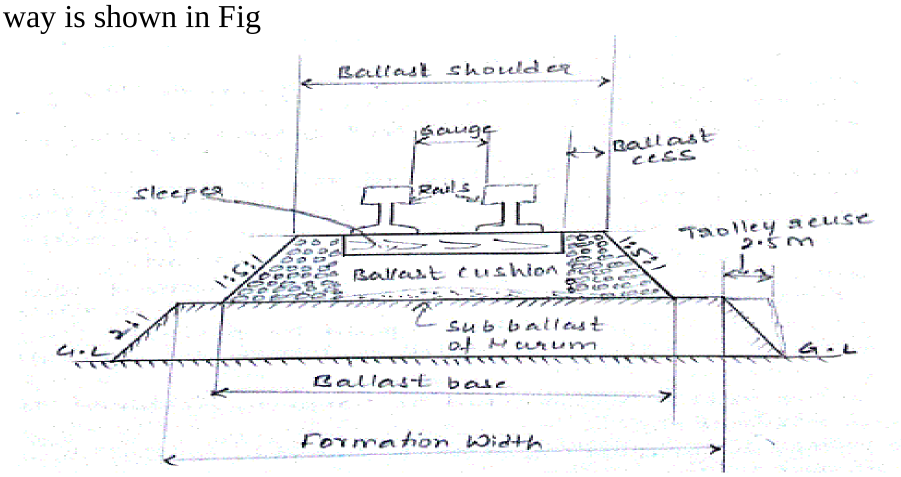


Fig 4.1 cross section of permanent way.

The Typical components are – Rails, Sleepers, Fasteners, Ballast (or slab track), Subgrade.

5.BASIC ELEMENTS OF AIRPORT:

Airport is a place, where aircrafts can take off as well landing operations are done. Usually they are equipped with hangers, facilities for refuelling and accommodation for passengers. The flow chart represents general classification of airports.

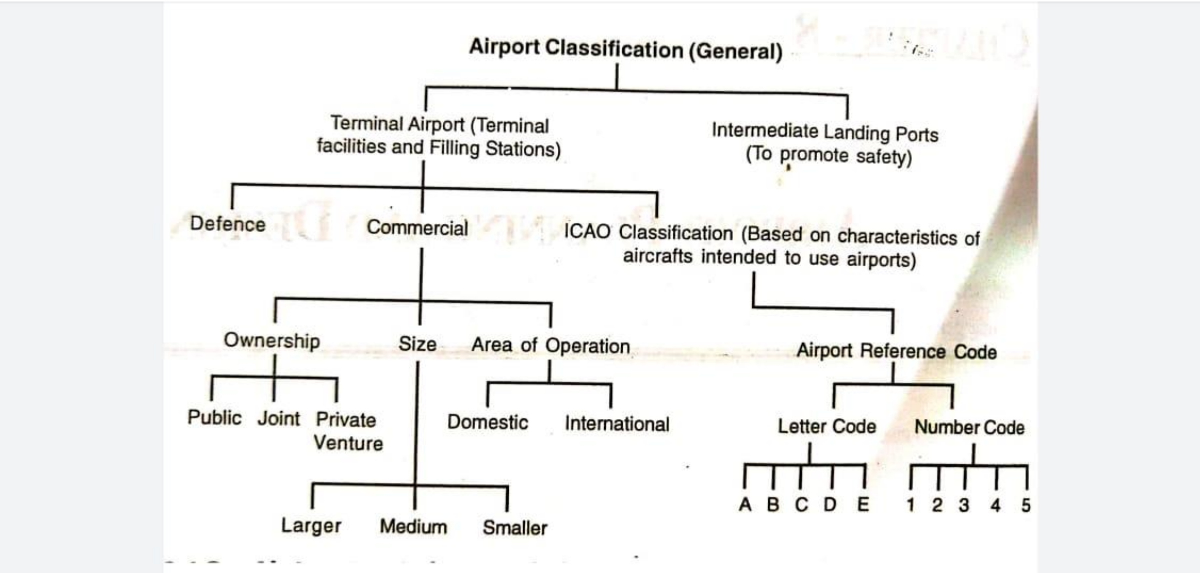


Fig 5.1 classification of airports

6.RUNWAY

Runway is a defined rectangular area prepared for landing and take-off of aircrafts and over which aircraft runs on ground. Runways are play a major role in arrangements of all components in the airport. Number of runways are depend on the volume of the air traffic & its orientation is depend on the WIND DIRECTION.

Generally, runways are connected to the taxiways because of

1. To avoid the delays in landing and take off operations
2. To provide shortest distance from terminal area to end of runways
3. To provide quick accessible for aircrafts to reach taxiways as early as possible.

To get actual runway length some considerations are taken into considerations. They are Correction of Elevation, Correction of Gradient, Correction of Temperature.

a)Correction of Elevation

- As per ICAO, the runway length should be increased at rate of **7% per 300 M** rise in elevation of about airport mean sea level.
- Air density reduces when elevation is increased.
- This shows effect on lift of aircraft while take off.

b)Correction of Gradient

- If the gradient become more steep, more consumption of energy takes place and require long runway length to attain ground speed.
- The maximum difference between highest and lowest points of runway divided by total length of runway is known as effective gradient.
- According to FAA(Federal Aviation Administration) of U.S.A**, runway length** after correction for temperature and elevation should be further increased **at rate of 20% for every 1%** of effective gradient.

c)Correction of Temperature.

- The elevation of airport is further increased at rate of **1% for every 1^{0} C** temperature.
- Airport reference temperature = $T_1 + (T_2 - T_1)/3$
- T_1=Monthly mean of average temperature for the hottest month of year
- T_2= Monthly mean of maximum daily temperature of the same month

7.TAXIWAY:

Taxiway main function is to provide access to the aircrafts from runways to the loading apron or service hanger and back. The following considerations decide the layout of taxiway.

- Taxiways are arranged that aircrafts which just landed and are taxing towards apron, do not interface with aircrafts taxing for take-off.
- At busy airports, taxiways should be located at various points along the runway so that the landing aircraft leaves the runway as early as possible and keeps it clear for other aircrafts. Such taxiways are called as Exit Taxiways.
- Exit taxiways should be designed for high turn off speeds, to reduce the runway occupancy time and thus increase the airport capacity.
- Taxiway route is shortest practicable distance from apron to runway end.
- Intersection of taxiway and runway should be avoided.

8.APRON:

It is a paved area for parking of aircrafts, loading and unloading of passengers and cargo. It is usually located close to the terminal building or hangers. The size of apron depends on:

(i)Size of loading area for each type of aircraft. This area is also known as Gate position.

(ii)Number of gate positions

(iii)Aircraft parking system

8.1.SIZE OF GATE POSITION:

Size of gate position is depend upon size of aircraft and its minimum turning radius.

The Manner in which aircraft enter and leaves the gate position under its own power or when pushed by a tractor.

Aircraft parking configuration also one of the main factor to decide the gate position. Parking of aircrafts caused least interference due to heat, fumes and blast. Jet engines are more critical than piston engines. The basic parking configurations is as follows:

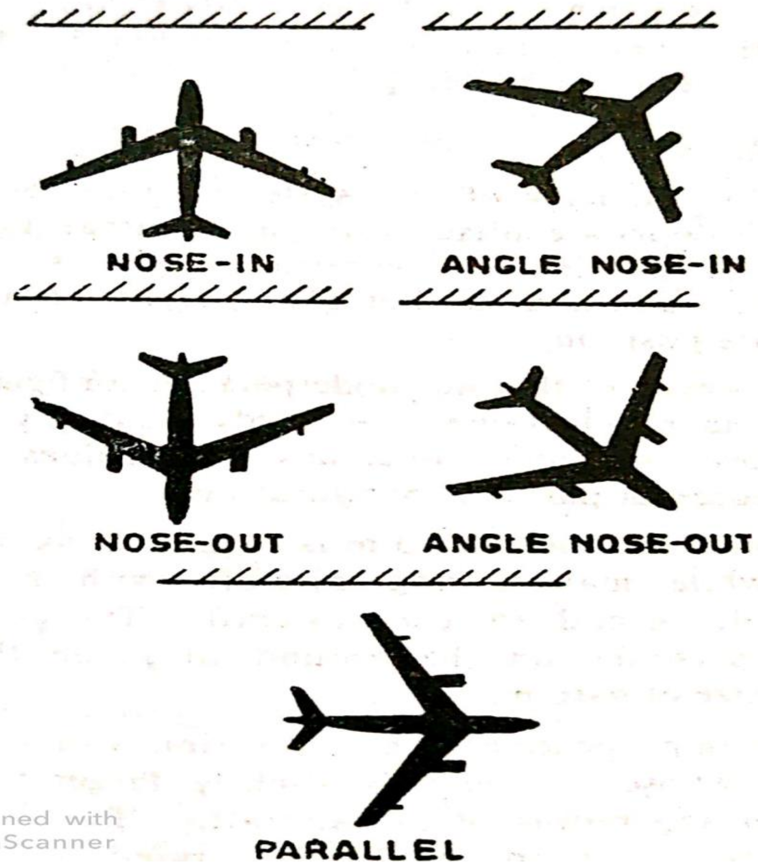


Fig 8.1.1 Parking position of aircrafts.

A) NOSE-IN AND ANGLED NOSE-IN

The **advantages** of this configuration are:

- Less noise while taxing in because no turning is required
- Hot blast is not directed towards the terminal building
- The aircraft forward door is close to the terminal building

The **disadvantages** are:

The aircraft rear loading door Is far away from terminal building.

(B)NOSE-IN AND ANGLED NOSE-OUT

The **advantages** of this configuration are as follows,

- Less power is required while manoeuvring the aircraft out of its gate position.
- The rear loading door is close to the terminal building.
- Overall apron area required is generally small.

The **main disadvantage** is that the hot blast is directed towards the terminal building.

(C)PARALLEL SYSTEM

The main **advantage** of this system is tbat,both,the front and the rear doors are adjacent to the terminal building. But this type of parking configuration requires more space. Further.the noise and the hot blast are directed towards the adjacent gate position.

Thus, it is evident that no single parking configuration can be considered as an ideal one. A minimum clearance of 7.5 m is suggested as the desirable clearance while manoeuvring aircraft with respect to terminal building and adjacent aircraft.

8.2.NUMBER OF GATE POSITIONS:

This mainly depends upon the peak hourly aircraft movements and the time

during which each aircraft remains in a gate position. This time in also known as the ramp time

and it varies from few minutes for small aircraft to more than an hour depending on the size. The required number of gate positions can be obtained from relationship,

Number of gate positions = (capacity of runway/60*2) * average gate occupancy time

Here runway 'capacity can.be taken in units of movements per hour. In the above formula, it is assumed that each aircraft occupying a gate position, represents two aircraft movements, landing and take off.But this may not always be so. The aircrafts are very often brought to the gate position from a service hangar. For the design, the gate occupancy time for big aircrafts may be assumed as 60 minutes. For small aircrafts requiring no. servicing, it may be assumed as 10 minutes.

8.3.AIRCRAFT PARKING SYSTEM:

Aircrafts can be grouped adjacent to terminal building in

various ways;

(i)Frontal system

(ii)Open apron system

(ii) Finger system

(iv) Satellite system

These parking systems are discussed below:

(i) FRONTAL SVSTEM: It is very simple and economical system. But its use is limited only to small airports requiring few gate positions.

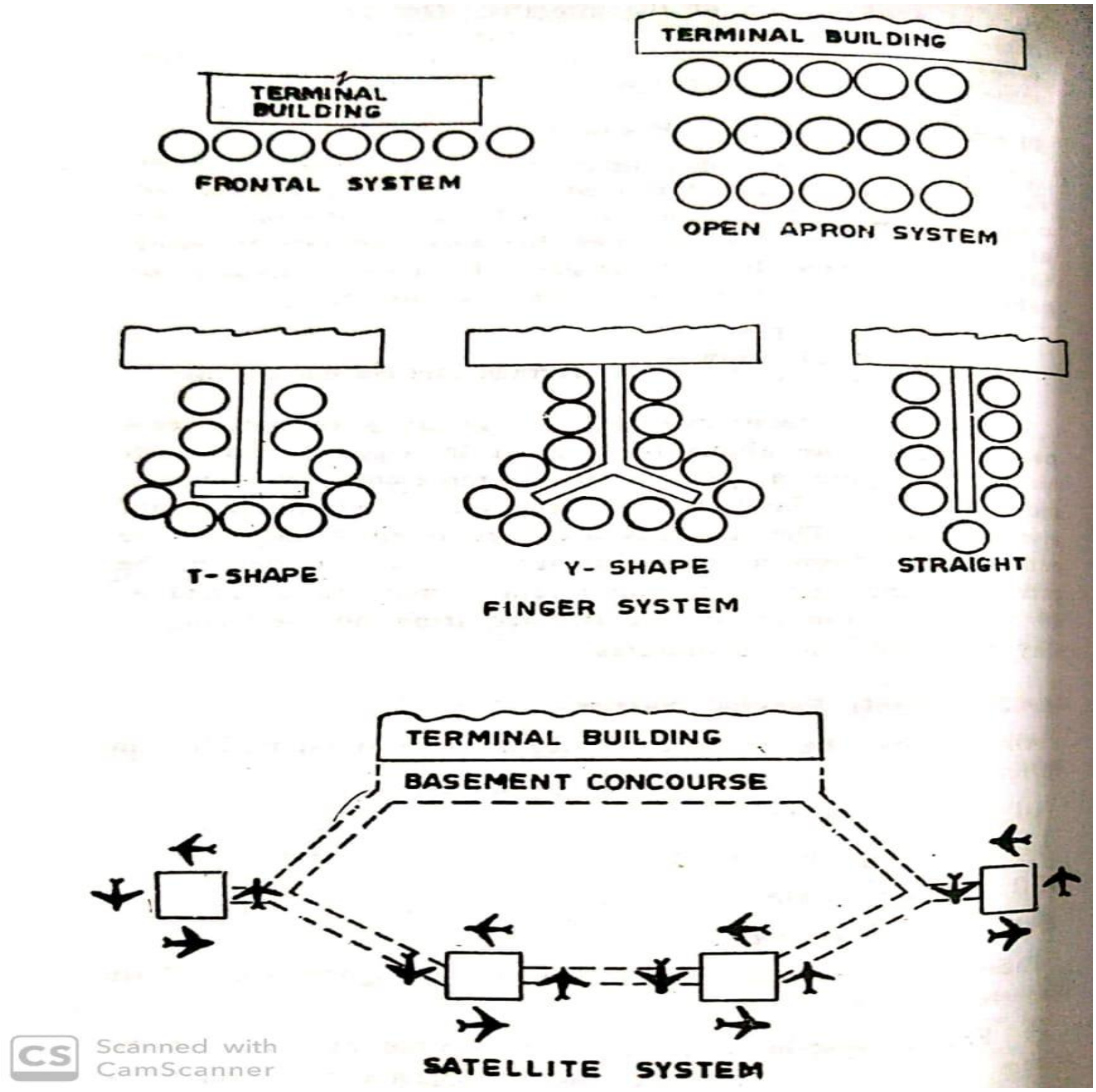


Fig 8.3.1 Parking system of aircraft.

(ii)OPEN APRON SYSTEM MARKED

In this system the aircrafts are parked in rows. If the number of aircrafts is too large, passengers may have to walk long distances or reach the aircrafts parked in the outermost row. They are thus exposed to weather, noise and hot blast of the jet aircrafts. To protect the passengers from such nuisance, some sort of closed vehicle conveyance for the passengers may be essential. The concept of conveying passengers from terminal building to the aircrafts by buses is in use in U.K. and U.S.A. But this is contrary to the objective of apron planning where it is desired to keep the vehicular traffic away to avoid hazards.

(iii) FINGER SYSTEM:

Processing of passengers and their But the facilities for passengers, for entering and leaving the aircraft, Such extension is known as **pier finger**. A typical arrangement is shown in Figure. The pier finger can be fenced open walkway or a closed structure, single or multi-storeyed. It can be a straight, T-shaped or Y-shaped. Its main advantages are:

- It provides adequate protection to the passengers from weather, noise, fumes etc. even when they come out of the terminal building.
- Future expansion is easier.
- All aircrafts remain close to the terminal building.

(d) It permits the installation of a short nose loading bridge or a swinging gang plank for the convenience of the passengers.

(iv)SATELLITE SYSTEM :

Satellites are small buildings located on the apron. Aircrafts are parked around the satellite buildings which are connected to the main terminal building by underground tunnel. This system is in use at the International pier finger system only when the connections to satellite airport of Los Angeles. It is advantageous, compared to the buildings are through the tunnels. In such an arrangement ,aircrafts are parked near the satellite as shown in Figure. Less turning is required to manoeuvre the aircraft and out of the gate position. The disadvantages of this system are (a) Large construction cost. (b) Passengers have to change the levels several times as they leave terminal building for boarding the aircraft.

9.TERMINAL BUILDING:

Terminal building usually refers to a building mainly, used to a building for passengers, airline and administration facilities. Its layout is such as to offer the enplaning passengers, the convenient and direct access from the vehicle platform or street side of the street side of the building through the booking and waiting rooms, to the waiting and rooms, to the aircraft loading positions on the apron. Deplaning passengers are also provided with a direct route from the aircraft to the baggage claim counter and then to the vehicle platform. The operational category includes control tower, weather bureau and other government services related to the aviation. In many cases the terminal building fulfils the function of the operational building as well.

The various facilities provided in the airport buildings are as follows:

(i)Passengers and baggage handling counters for booking

(ii) Baggage claim section

(iii) Enquiry counter

(iv) Space for handling and processing mail, express and light cargo

(v) Public telephone booth

(vi) Waiting hall for passengers and visitors

(vii) Toilet facilities

(viii)Restaurant and bars

(ix)First aid room

(x)General store and gift shops

(xi)Office space for airport staff

(xii)Passport and health control etc..,

9.1.PLANNING CONSIDERATIONS:

In planning considerations, two concepts are there for planning of the terminal buildings for a commercial airport. Centralization and Decentralization In the centralized plan, all passengers, baggage and cargo are funnelled through a central building and are then dispersed to the respective aircraft positions. In the decentralised plan, the passengers and baggage arrive at a point near the departing plane. All airline functions are carried out adjacent to the departing plane. The choice of a particular type of plan is governed by the space needed for parking of the aircrafts. When the aircraft parking area is located at an overall walking distance exceeding 180 m, a change from the centralized system becomes necessary. Further, when the number of gate positions (loading area required for each aircraft) required for the individual airliner at one airport exceeds the decentralized plan also becomes operationally uneconomical. At this situation, another shift towards the centralization of each individual airline operation becomes essential. This of results in a series of centralized airline spaces, arranged in a decentralized pattern.

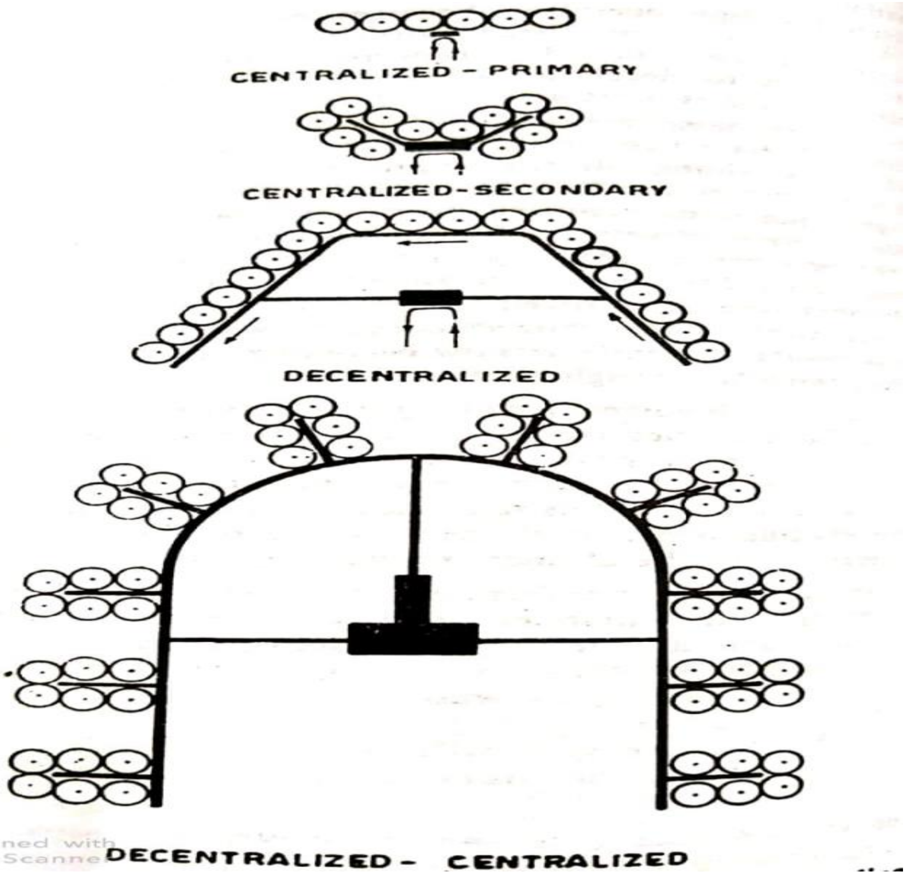


Fig 9.1.1 Centralized & decentralized systems.

10.HANGAR:

The primary function of a hangar is to provide an enclosure for servicing, overhauling and doing repairs of the aircrafts. They are usually constructed of steel frames and covered with galvanised iron sheets. They are also provided with machine shops and stores for spare parts. The size of hangar depends upon the size of aircraft and its turning radius. Adequate lighting inside the hangar is of prime importance. Sometime ceilings of hangar and some portions of its side walls are glazed, which work as light reflectors. Construction a hangar to store large number of aircrafts may be undesirable both from economy and other considerations viz., difficulty in the manoeuvring of aircrafts, noise nuisance, fumes, fire hazards etc. The number of hangars depends upon the peak hours volume of aircrafts and demand of hangars on rental basis by different airline agencies. The requirements of suitable hangar site are as follows

(i) The site should be such that there is a convenient road access to it from the site to the aprons and terminal buildings.

(ii) Proximity to and easy installation of utilities, such as electricity, telephone, water supply and sewers etc.

(iii) Reasonable proximity to the loading apron

.(iv) The site should not be along the direction of frequent storms as this is likely to damage the hangar doors etc.

(v) Sufficient area to provide car parking facilities for working personnel.

(vi) Favourable topography providing good natural drainage.

(vii) Adequate site area for future expansion of hangar facilities.

11.AIRCRAFT CHARACTERISTICS

The following characteristics need to be studied

1 Type of propulsion

2 Size of aircraft

3 Minimum turning radius

4 Minimum circling radius

5 Speed of aircraft

6 Capacity of aircraft

7 Aircraft weight and wheel configuration

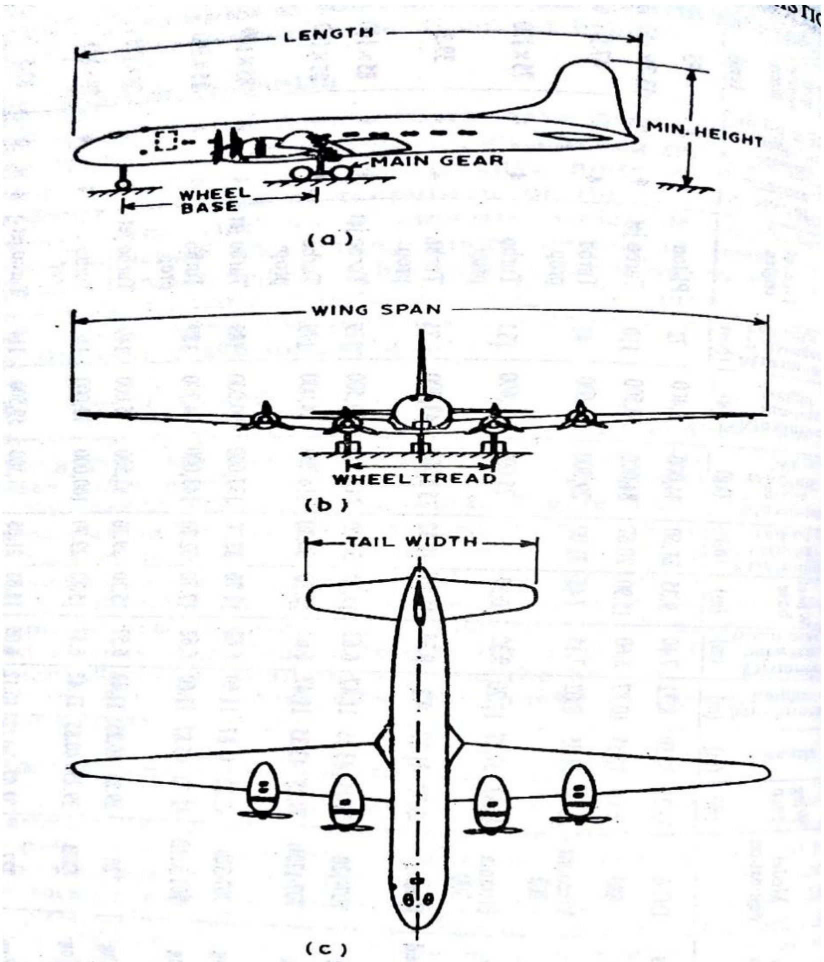
8 Jet blast

9 Fuel spillage

10 Noise

1.Types of Propulsion The size of aircraft, its circling radius, speed characteristic, weight carrying capacity, noise nuisance etc. depend upon the type of propulsion of the aircraft. ν The performance characteristics of aircrafts, which determine the basic runway length, also depend upon the type of propulsion. That heat nuisance due to exhaust gases is a characteristic of turbo jet and turbo prop engines

2.Size of Aircraft The sizes of aircraft involves following important dimensions: (i) Wing span (ii) Fuselage length (iii) Height (iv) Distance between main gears, i.e. gear tread (v) Wheel base and (vi) Tail width. These are shown in Figure 6.3. The wing span decides the width of taxiway, separation clearance between two parallel traffic ways, size of aprons and hangars, width of hangar gate etc. The length of aircraft decides the widening of taxiways on curves width of exit taxiway, sizes of aprons and hangars etc. The height of aircraft, also called as empennage height, decides the height of hangar gate and miscellaneous installations inside the hangar. The gear tread and the wheel base affect the minimum turning radius of the aircraft.



a. Side view b. Front view and c. Plan
Fig.6.3 Aircraft dimensions

3.Minimum Turning Radius In order to decide the radius of taxiways, the position of aircrafts in loading aprons and hangars and to establish the path of the movement of aircraft, it is very essential to study the geometry of the turning movement of aircrafts. The turning radius of an aircraft is illustrated in the Figure 6.4. To determine the minimum tuning radius, a line is drawn through the axis of the nose gear when it is at its maximum angle of rotation. The point, where this line intersects another line drawn through the axis of the two main, gears, is called the centre of rotation. The distance of the farther wing tip from the centre of rotation represens the minimum turning radius. Theoretically, the maximum angle of rotation is 90°. Corresponding to this, turning radius would be absolute minimum, the condition which causes the skidding of one of the main gears there by producing excessive wear. To keep the tire wear of the main gears within reasonable limits, the maximum angle of rotation of the nose gear has been limited by the manufactures.

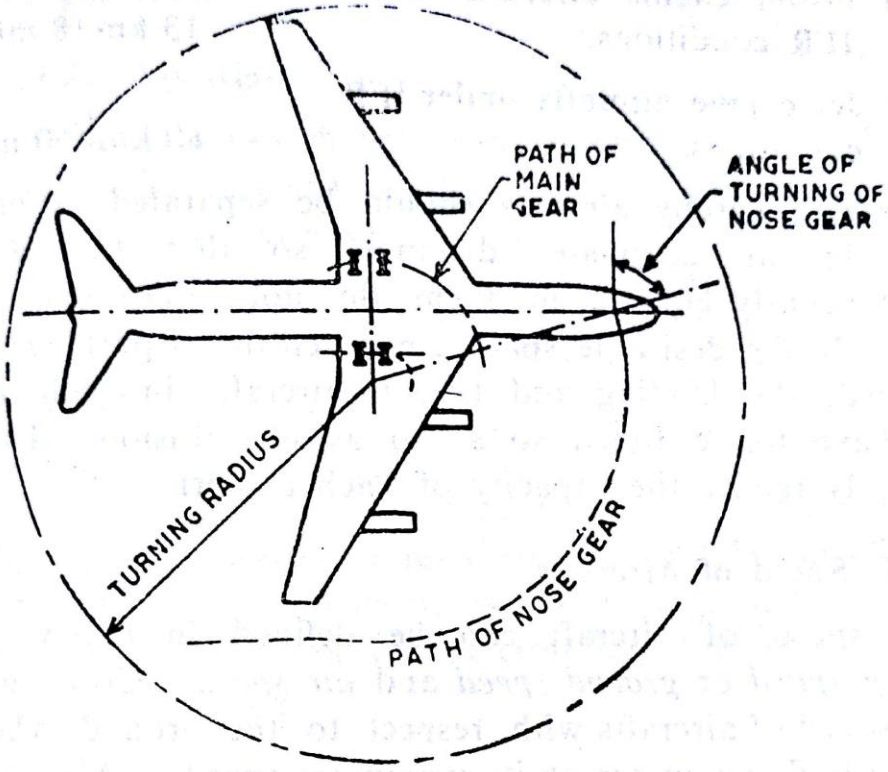


Fig.6.4 Turning radius of aircraft

4.Minimum Circling Radius There is certain minimum radius with which the aircraft can take turn in space. This radius depends upon the type of aircraft air traffic volume and weather conditions. The radii recommended for different types of aircrafts are as follows ν (i) Small general aviation aircrafts under UFR conditions, 1.6 km (1 mile) (ii) Bigger aircrafts, say two piston engine under VFR conditions = 32 km (2 mile) (iii) Piston engine aircrafts under IFR conditions. = 13 kin (8 miles) (iv) Jet engine aircrafts under IFR conditions= 80 km (50 mites) The two nearby airports should be separated from each other by an adequate distance so that the aircrafts simultaneously landing on them do not interfere with each other. If the desirable spacing between the airports cannot he provided, the landing and take-off aircrafts in each airport will have to be timed so as to avoid collision.

5.Speed of Aircrafts The speed of aircraft can be defined in two ways viz. cruising speed or ground speed and air speed. Cruising speed is the speed of aircrafts with respect to the ground when the aircraft is flying in air at its maximum speed. Air speed is the speed of aircraft relative to the wind. Thus, if the aircraft is flying at a speed of 500 kmph and there is a head wind of 50 kmph, air speed will be 450 kmph.

6.Aircraft Capacity The number of passengers, baggage, cargo and fuel that can be accommodated in the aircrafts depends upon the capacity of aircraft. The capacity of aircraft using an airport has an important effect on the capacity of runway systems as well as that of the passenger processing terminal facilities.

7.Weight of Aircraft & Wheel Configuration Weight of the aircraft directly influence the length of the runway as well as the structural requirements i.e. the thickness of the runway, taxiway, apron & hangars. It depends not only on the weight of the passenger baggage, cargo and fuel it is carrying and its structural weight, but also on the fuel which is continuously decreasing during the course of the flight.

8.Jet Blast At relatively high velocities, the aircrafts eject hot exhaust gases, The velocity of jet blast may be as high as 300 kmph. This high velocity cause inconvenience to the passengers traveling in the aircraft. Several types of blast fences or jet blast deflector are available to serve as an effective measure for diverting the smoke ejected by the engine to avoid the inconvenience to the passengers. Since, the bituminous (flexible) pavements are affected by the jet bust, therefore, it is desirable to provide cement concrete pavement at least at the touch down portion to resist the effect of the blast in preference to the bituminous pavements. The effect of the jet blast should also be considered for determining the position, size and location of gates.

9.Fuel Spillage At loading aprons and hangars, it is difficult to avoid spillage completely, but effort should be made to bring it within minimum limit. The bituminous (flexible pavements are seriously affected by the fuel spillage and therefore, it is essential that the areas of bituminous pavements under the fuelling inlets, the engines and the main landing gears are kept under constant supervision by the airport authorities.

10.Noise generated by aircraft creates problems in making decisions on layout and capacity. The correct assessment of future noise patterns to minimize the effect of surrounding communities is essential to the optimal layout of the runways. The FAA noise regulations came into force in 1969 for jet-powered aircraft with bypass ratios greater than 2. In 1973, they were modified to apply to all aircraft manufactured after that date.

12.HARBOUR ENGINEERING:

12.1.PORT:

A port is a gateway to land from the sea and from sea to the land. It is a point of change from land carriage to sea carriage. Ships bring passengers and goods from overseas and discharge them in the port for convergence to inland destinations. A port is a commercial harbour with all infrastructures.

12.2.TYPES OF PORTS:

1.SEA PORT: The port used to handle ocean-going is called as sea port.

2.RIVER PORT: The river traffic port such as shallow draft vessels and barges are controlled by river port.

3.FISHING PORT: Fishing port is distributing and storey of the fishes.

4.WARM WATER PORT: the place where there is no freeze of water in winter is called as warm water port.

5.DRY PORT: A place where containers or conventional bulk cargo is called dry port. Normally, it is connected to a sea port by road or rail.

6.INLAND PORTS: the direct access of ports on river, lake or canal to ocean or sea is called as inland port.

13.HARBOUR:

Harbour can be defined as a basin or haven or road-stead of navigable waters well protected naturally or artificially from action of wind and waves, and is situated along sea-shore or river estuary or lake or canal connected to sea. Draft is vertical linear immersion of ship below water surface for the ship to float in stable condition safely. Safe floating requires a standard vertical clearance between bottom part of ship and sea bed. Harbours situated on mouth of river are know as river harbours, while those situated along lake are known as lake harbours, those situated along canals are known a canal harbours. River harbours have problems of continuous deposition which if not overcome by maintenance dredging navigability, is likely to be lost. As methods of navigation improved, these vessels increased in size, number and importance.

The harbours are classified as under

1) Classification depending upon the protection

2) Classification depending upon the utility

3) Classification based upon the location.

(1)CLASSIFICATION OF HARBOUR DEPENDING UPON THE PROTECTION NEEDED:

Depending upon the protection needed harbours are broadly classified as:

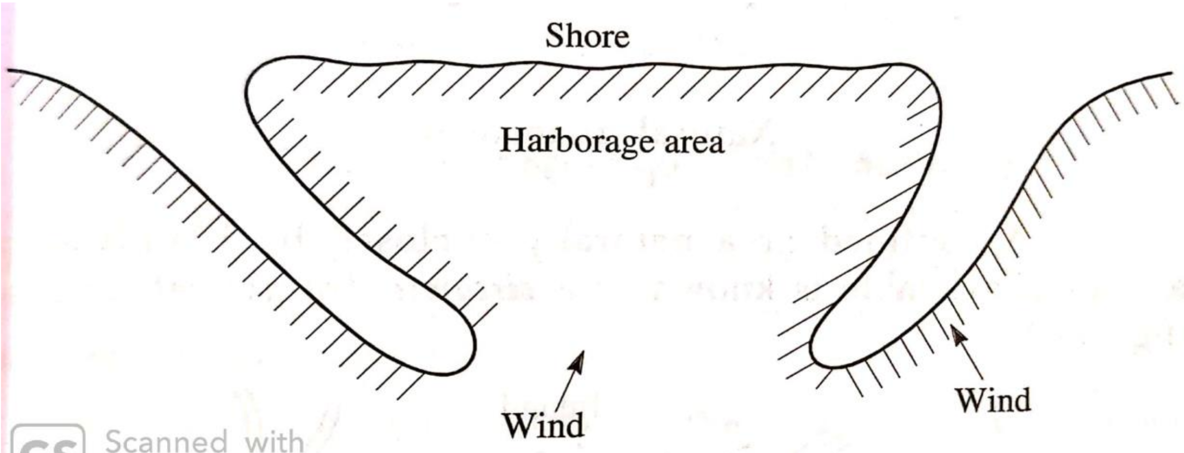
(i) Natural harbours.

(ii) Semi-natural harbours

(iii) Artificial harbours

(i)NATURAL HARBOURS:

Natural formations affording safe discharge facilities for ships on sea coasts, in the form of creeks and basins, are called natural harbours. In other words, natural harbour is an inlet protected from storms and waves by natural configuration of land. With the rapid development of navies engaged either in commerce or war, improved accommodation and facilities for repairs, storage of cargo and connected amenities had to be provided in natural harbours. The size and draft of vessels have necessitated the works of extension and improvement for natural harbours. The factors such as local geographical features, growth of population, development of the area, etc. have made the natural harbours big and attractive. Bombay and Kandla are examples of natural harbours.

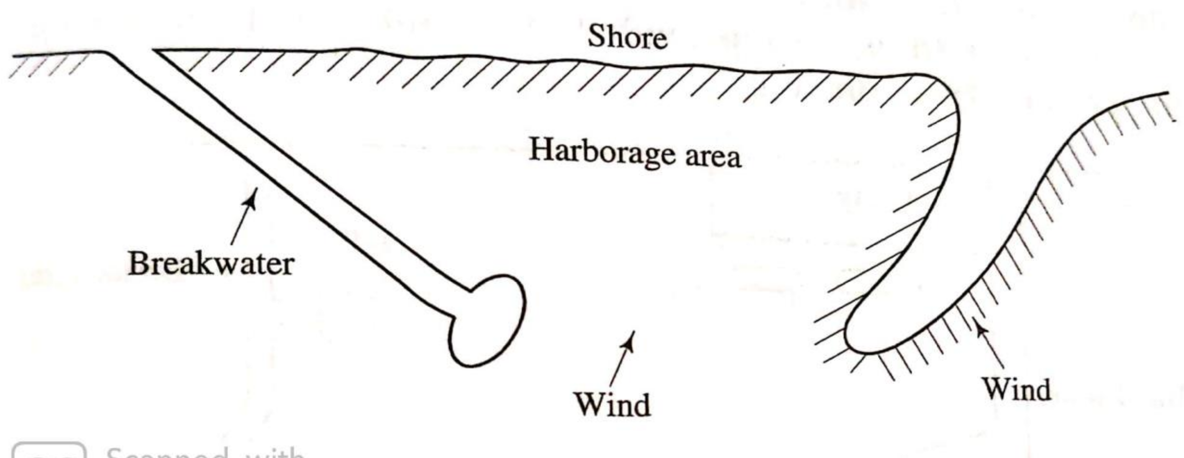


Natural harbour

Fig 13.1 Natural harbour

(ii) SEMI-NATURAL HARBOURS:

This type of harbour is protected on sides by headlands and it requires man-made protection only at entrance as shown in following figure. Visakhapatnam is a semi-natural harbour.



Semi-natural harbour

Fig.13.2 semi-natural harbour

(iii) ARTIFICIAL HARBOUR:

Artificial harbours are where natural facilities are not available, countries having a sea board had to construct such shelters making use of engineering skill and methods. Such harbours are called as ARTIFICIAL OR MAN MADE HARBOURS. It is an area protected from effect of waves. Madras harbour is an example of artificial harbour.

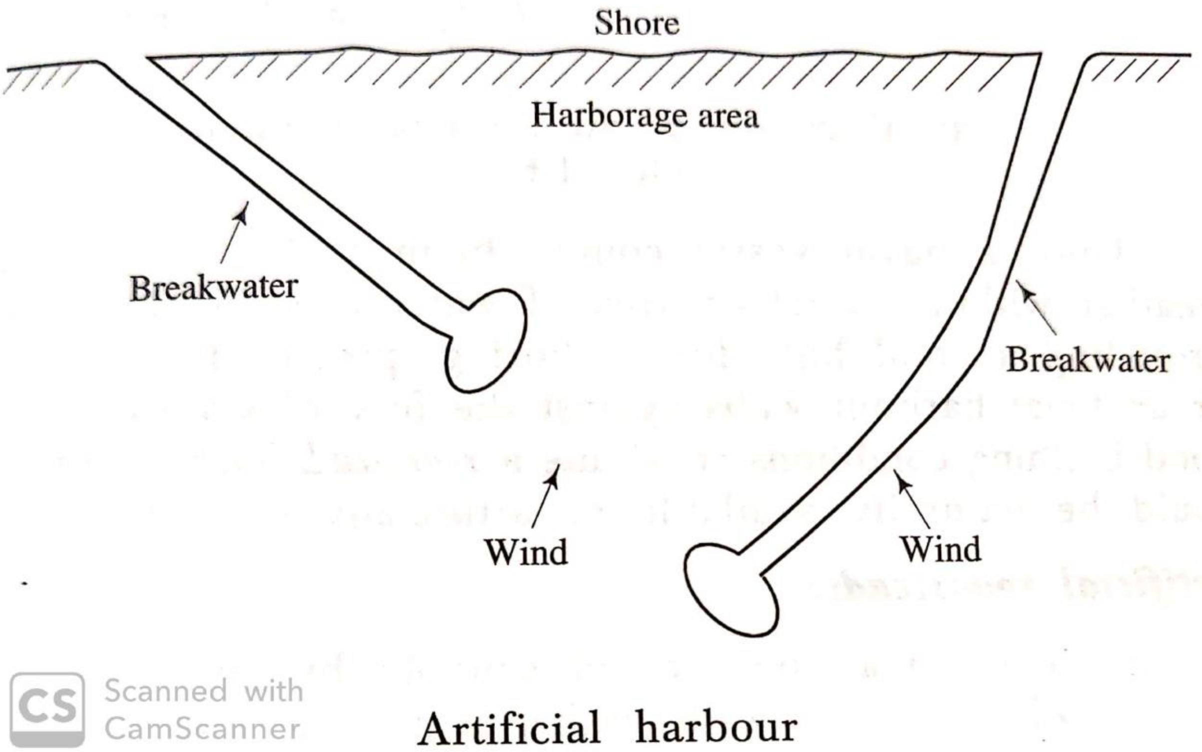


Fig 13.3 Artificial harbour

(2)CLASSIFICATION DEPENDING UPON THE UTILITY:

From their utility, harbours are further into five major types

(i)Harbours of refuge

(ii) Commercial harbours

(iii) Fishery harbours

(iv) Military harbours

(v) Marina harbours

It is necessary to study the requirements of these types of harbours and provide for such requirements.

(i) Harbours of refuge:

Requirements of harbour of refuge

(i) Ready accessibility from the high seas.

(ii)Safe and Convenient anchorage against the sea.

(iii) Facilities for obtaining supplies and repairs.

On dangerous coast-lines, disabled or damaged ships under stress of weather conditions will need quick shelter and immediate repairs. All types of naval craft small and big will need such refuge in emergency and hence refuge harbours should provide commodious accommodation. Modern big ships will require a lot of elbow room for purpose of manoeuvring or turning about.

(ii) COMMERCIAL HARBOURS:

Requirements of commercial harbour:

(i)Spacious accommodation for the mercantile marine.

(ii Ample quay space and facilities for transporting,()loading and unloading cargo.

(iii) Storage sheds for cargo.

(iv Good and quick repair facilities to avoid delay.

(v) More sheltered conditions as loading and unloading could be done with advantage in calmer waters.

Commercial harbours could be situated on coasts or estuaries of big rivers or even on inland river coasts. They do not normally have any emergency demand like a harbour of refuge and practically the size and number of ships using such harbours are known factors.

(iii) FISHERY HARBOURS:

Requirements of fishery harbour:

(i)Harbour should be constantly open for departure and arrival of fishing ships.

(ii) Loading and unloading facilities and quick despatch facilities for the perishable fish catch like railway sidings and roads.

(iii) Refrigerated stores with ample storing space for preserving the catch.

(iv) MILITARY HARBOURS:

Requirements of military harbour:

These harbours are the naval bases which are meant to accommodate the naval vessels. They serve as supply depots also. Bombay and Cochin harbours have naval bases.

(v)MARINA HARBOURS:

Marina is harbour providing facilities of fuel, food, showers, washing machines, telephone, etc. for Small boat owners, having temporary or permanent berths. These are classified in two categories:

(a) Large marinas (b) Small marinas.

(a) LARGE MARINAS:

The large marinas have 200 or more berths and cater large boats. At times space available is limited and hence create a long waiting list to get berth.

(b) SMALL MARINAS:

The small marinas have less than 100 berths. Marinas in general are located on fresh waters or on coastal waters.

(3) CLASSIFICATION OF HARBOUR BASED UPON THE LOCATION:

The layout of a harbour is greatly influenced by its location and based on the location, harbours are further classified into the following four major types:

(i) Canal harbour

(ii) Lake harbours

(iii) River or estuary harbour

(iv) Sea or ocean harbour.

(I)CANAL HARBOUR:

The harbour located along the canals for sea navigations and inland, is known as canal harbour. It is found that the maintenance dredging of canal harbour basins is generally negligible.

(II) LAKE HARBOUR:

The harbour constructed along the shore of lake is known as lake harbour. If the lake is large the conditions are similar to those in ocean except that tidal action does not occur.

(III) RIVER OR ESTUARY HARBOUR:

The harbour constructed along the banks of river is known as river or estuary harbour. Rivers and estuaries create the main transportation route to join the hinterland and the sea. The best possibilities for sea-going navigation are offered by the lower reaches of a river where the tides are determining the hydraulic conditions. Hence, many sea-ports have been Constructed on a tidal river.

(IV) SEA OR OCEAN HARBOUR:

The harbour located on the coast of a sea or an ocean is called the sea harbour. They are intended for sea-going vessels.

14.REQUIREMENTS OF A GOOD PORT:

These can be enlisted as follows:

It should be situated for the hinterland. For a port, the hinterland is that part of the country behind which it can be served with economy and efficiency by the port.

(ii) It should get good tonnage i.e. charge per KN of cargo handles by it.

(iii) It should have good Communication with the rest of the country through rail and highways so that the commodities can be transported to and from the port easily and quickly.

(iv) The hinterland should be fertile with a good density of population.

(v) It should be advanced in culture, trade and industry.

(vi) It should be a place of defence and for resisting the sea-born invasion.

(vii) It should command valuable and extensive trade.

(viii) It should be capable of easy, smooth and economic development.

(ix) It should afford shelter to all ships and at all seasons of the year.

(x) It should provide the maximum facilities to all the visiting ships including the servicing of ships.

(xi) The passage to open sea must have sufficient depth and width and it should be suitably marked to aid navigation.

(xii) The land surfaces of the coastline should be fully hard so that frequent repairs are not required. If the coast is sandy, intermittent repairs to docks and port buildings will have to be carried out frequently making their maintenance very expensive.

UNIT II

HIGHWAY DEVELOPMENT AND PLANNING

Objective:

To familiarize with different concepts in the field of Highway Engineering

Syllabus: Highway Development and Planning

Introduction about Roads, Jayakar Committee and its recommendations, Necessity for Highway Planning, Different Road Development plans, Classification of Roads, Road network patterns, Highway Alignment, Factors affecting Alignment, Engineering Surveys.

Learning Outcomes:

After completion of this unit the student will be able to

- plan the alignment of highway network for the given area.

Learning Material

1. Modern developments

The First World War period and that immediately following it found a rapid growth in motor transport. So need for better roads became a necessity. For that, a resolution was passed by both chambers of the Indian Legislature 1927 for appointment of a committee to examine and report on the question of road development in India. In response to the resolution, India Road Development Committee was appointed by the government with Mr.M.R. Jayakar as the chairman, in 1927.

1.1 Jayakar Committee

The Jayakar committed submitted its report by the year 1928. The most important recommendations made by the committee are:

i. The road development in the country should be considered as a national interest as this has become beyond the capacity of provincial governments and local bodies.

ii. An extra tax should be levied on petrol from the road users to develop a road development fund called Central Road Fund (CRF).

iii. A semi-official technical body should be formed to pool technical knowledge from various parts of the country and to act as an advisory body on various aspects of roads.

iv. A research organization should be instituted to carry out research and development work and to be available for consultations.

Most of the recommendations of the Jayakar Committee were accepted by the government, and the major items were implemented subsequently. The Central Road fund was formed by the year 1929, the semi-official technical body called the Indian Roads Congress was formed in 1934 and the Central Road Research Institute was started in 1950.

1.1.1 Central Road Fund

Based on the authority of a resolution adopted by the Indian Legislature, the Central Road Fund (C.R.F) was formed on 1st march 1929. The consumers of petrol were charged an extra leavy of 2.64 paisa per litre (then two annas per gallon) of petrol to build up this road development fund 20 percent of the annual revenue is to be retained as a Central Reserve, from which grants are to be given by the Central Government for meeting expenses on the administration of the road fund, road experiments and research on road and bridge projects of special importance. The balance 80 percent is to be allotted by the Central Government to the various states based on actual petrol consumption or revenue collected.

Central road fund act-2000 was notified in December 2000 which gave statutory status to the existing central road fund governed by a resolution of the parliament in 1988.At present the revised cess collected on petrol and HSD towards CRF is @ Rs2/-.

1.1.2 INDIAN ROADS CONGRESS

- A semi-official technical body known as Indian Roads Congress (IRC) was formed in 1934. This is one of the main recommendations made by the Jayakar Committee.
- The Indian Roads Congress was constituted to provide a forum for regular pooling of experience and ideas on all matters affecting the planning, construction and maintenance of roads in India, to recommend standard specifications and to provide a platform for the expression of professional opinion on matters relating to road engineering including such questions as those of the three 20-year development plans in India.

- Now the Indian Roads Congress has become an active body of national importance controlling specifications, standardization and recommendations on materials, design and construction of roads and bridges.
- The IRC publishes journals, research publications, standards specifications guidelines and other special publications on various aspects of Highway Engineering. The technical activities of the IRC are mainly carried out by the Highway research Board and several close collaboration with Roads Wing of the Ministry of Surface Transport, Government of India.

1.1.3 MOTOR VEHICLE ACT

In 1939 the Motor Vehicles Act was brought into effect by Government of India to regulate the road traffic in the form of traffic laws, ordinances and regulations. The three phases primarily covered are control of the driver, vehicle ownership and vehicle operation on roads and in traffic stream. The Motor Vehicle Act has been appended with several ordinances subsequently. The Motor Vehicles Act has been revised in the year 1988.

2.Different Road Development plans

2.1Nagpur road congress 1943-63

The Second World War saw a rapid growth in road traffic and this led to the deterioration in the condition of roads. To discuss about improving the condition of roads, the government convened a conference of chief engineers of provinces at Nagpur in 1943. The result of the conference is famous as the Nagpur plan.

A twenty year development programme for the period (1943-1963) was finalized. It was the first attempt to prepare a co-ordinated road development programme in a planned manner.

The roads were divided into four classes:

1. National highways which would pass through states, and places having national importance for strategic, administrative and other purposes.

2. State highways which would be the other main roads of a state.

3. District roads which would take traffic from the main roads to the interior of the district. According to the importance, some are considered as major district roads and the remaining as other district roads.

4. Village roads which would link the villages to the road system.

- 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.
- One of the objective was that the road length should be increased so as to give a road density of 16kmsper 100 sq.km

2.2 Bombay road congress 1961-81

The length of roads envisaged under the Nagpur plan was achieved by the end of it, but the road system was deficient in many respects. The changed economic, industrial and agricultural conditions in the country warranted a review of the Nagpur plan. Accordingly a 20-year plan was drafted by the Roads wing of Government of India, which is popularly known as the Bombay plan. The highlights of the plan were:

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

2.3 Lucknow road congress 1981-2001

This plan has been prepared keeping in view the growth pattern envisaged in various fields by the turn of the century. Some of the salient features of this plan are as given below:

- This was the third 20 year road plan (1981-2001). It is also called Lucknow road plan.
- It aimed at constructing a road length of 12 lakh kilometres by the year 1981 resulting in a road density of 82kms/100 sq.km
- The plan has set the target length of NH to be completed by the end of seventh, eighth and ninth five year plan periods.
- It aims at improving the transportation facilities in villages, towns etc. such that no part of country is farther than 50 km from NH.

- One of the goals contained in the plan was that expressways should be constructed on major traffic corridors to provide speedy travel.
- Energy conservation, environmental quality of roads and road safety measures were also given due importance in this plan.

3. NECESSITY OF HIGHWAY PLANNING

In the present era planning is considered as a pre-requisite before attempting any development programme. This is particularly true for any engineering work, as planning is the basic requirement for any new project or an expansion programme. Thus highway planning is also a basic need for highway development. Particularly planning is of great importance when the funds available are limited whereas the total requirement is much higher. This is actually the problem in all developing countries like India as the best utilization of available funds has to be made in a systematic and planned way.

The objects of highway planning are briefly given below:

(i) To plan a road network for efficient and safe traffic operation, but at minimum cost. Here the costs of construction, maintenance and renewal of pavement layers and the vehicle operation costs are to be given due consideration.

(ii) To arrive at the road system and the lengths of different categories of roads which could provide maximum utility and could be constructed within the available resources during the plan period under consideration?

(iii) To fix up date wise priorities for development of each road link based on utility as the main criterion for phasing the road development programme.

(iv) To plan for future requirements and improvements of roads in view of anticipated developments.

(v) To work out financing system.

4. Classification of Roads

4.1 Methods of classification of roads:

1. ***Traffic volume***: The classification based on traffic volume or tonnage has been arbitrarily fixed by different agencies and there may not be a common agreement regarding the limits for each of classification group. Based on the traffic volume, the roads are

classified as heavy, medium and light traffic roads. These terms are relative and so the limits under each class should be clearly defined and expressed as vehicle per day etc.

2. ***Load transported or tonnage***: The classification based on load or tonnage is also relative and the roads may be expressed as tonnes per day and they are classified as I,II or class A,B
3. ***Location and function***: This is more acceptable classification of roads.

There are different approaches for road classification

4.2 Classification based on whether they can be used different seasons:

a) All-weather roads: All weather roads are those which are negotiable during all weather, except at major river crossings where interruption to traffic is permissible up to a certain extent, the road pavement should be negotiable during all weathers.

b) Fair-weather roads: Roads which are called fair weather roads; on these roads. The traffic may be interrupted during monsoon season at causeways where streams may overflow across the road.

4.3 Based on type of carriage way:

(i) Paved roads: If they are provided with a hard pavement course which should be at least a water bound macadam (WBM) layer and

(ii) Unpaved roads: If they are not provided with a hard pavement course of at least a WBM layer. Thus earth roads and gravel roads may be called unpaved roads. Ex. Gravel and Earth roads

4.4 Based on type of pavement surfacing provided:

(i) ***Surface roads***: Which are not provided with bituminous or cement concrete surfacing and

(ii***) Unsurfaced roads***: Which are not provided with bituminous or cement concrete surfacing. The roads provided with bituminous surfacing are also called black toped roads.

4.5 Based on location and function (Nagpur road plan)

- National highway (NH)
- State highway (SH)
- Major district road (MDR)
- Other district road (ODR)

- Village road (VR)

4.5.1 National Highways

- NH are the main highways running through the length and breadth of India, connecting major parts, foreign highways, capital of large states and large industrial and tourist centers including roads required for strategic movements for the defense of India.
- The national highways have a total length of 70,548kms. Indian highways cover 2% of the total road network of India and carry 40% of the total traffic.
- The highway connecting Delhi-Ambala-Amritsar is denoted as NH-1, where as a bifurcation of this highway beyond Jalandar to Srinagar and Uriis denoted NH-1-A.
- The longest highway in India is NH7 which stretches from Varansi in UttarPradesh to Kanya kumara in the southern most point of Indian main land.
- The shortest highway is NH47A which stretches from Ernakulamto Kochi and covers total length of 4 Kms.
- Golden Quadrilateral –(5,846 Kms) connecting Delhi-Kolkata-Chennai-Mumbai
- NH-2 Delhi-Kol(1453 km)
- NH 4,7&46 Che-Mum (1290km)
- NH5&6 Kol-Che(1684 m)
- NH 8 Del-Mum (1419 km)

4.5.2 State Highways

- They are the arterial roads of a state, connecting up with the national highways of adjacent states, district headquarters and important cities within the state.
- Total length of all SH in the country is 1, 37,119 Km.
- Speed 80 kmph.

4.5.3 Major District Roads

- Important roads with in a district serving areas of production and markets, connecting those with each other or with the major highways.
- India has a total of 4,70,000 km of MDR.
- Speed 60-80kmph

4.5.4 Other district roads

- Serving rural areas of production and providing them with outlet to market centers or other important roads like MDR or SH.
- Speed 50-60 kmph

4.5.5 Village roads

- They are roads connecting villages or group of villages with each other or to the nearest road of a higher category like ODR or MDR.
- India has 26, 50,000 km of ODR+VR out of the total 33,15,231 km of all type of roads.
- Speed 40-50 kmph

4.6 Modified classification based on Lucknow plan:

Roads are classified into three classes for the purpose of transport planning, functional identification, earmarking administrative jurisdictions and assigning priorities on the road network.

i. Primary system
ii. Secondary system and
iii. Tertiary system

Primary system:

It include Expressways and National highways

Expressways: have superior facilities and design standards and have high volume of traffic. They are provided with the divided carriageways, controlled access and grade separators.

Expressways

•Heavy traffic at high speed (120km/hr)

•Land Width (90m)

•Full access control

•Connects major points of traffic generation

•No slow moving traffic allowed

•No loading, unloading, parking

Secondary system: State highways and Major district roads

Tertiary system: Other district roads, village roads

4.7 Urban Road Classification

- Arterial Roads
- Sub Arterial
- Collector
- Local Street

5. Road Patterns

- Rectangular or Block patterns
- Radial or Star block pattern
- Radial or Star Circular pattern
- Radial or Star grid pattern
- Hexagonal Pattern
- Minimum travel Pattern

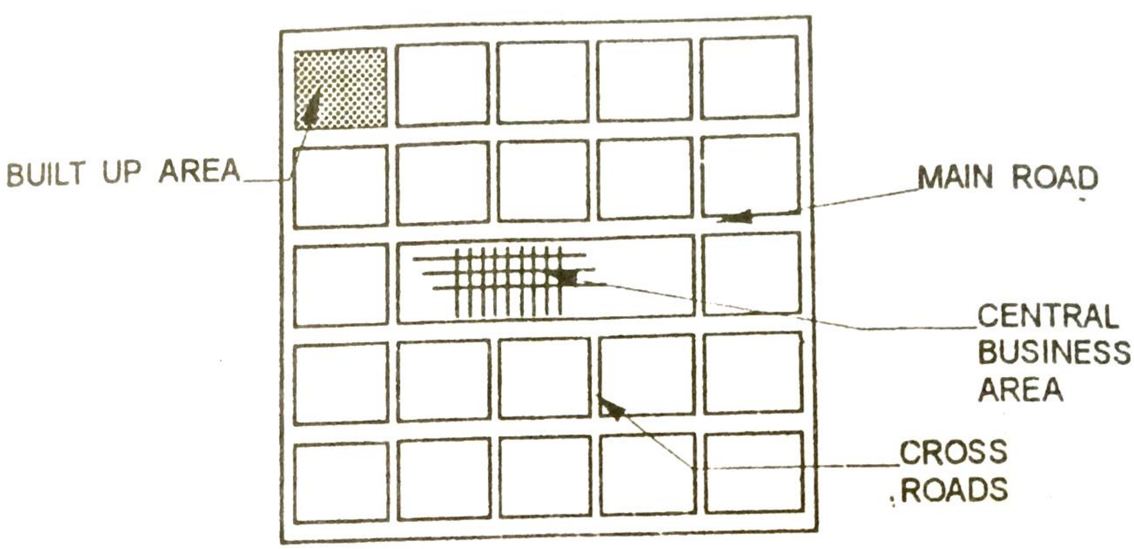


Fig. 4.1(a) Rectangular or block pattern

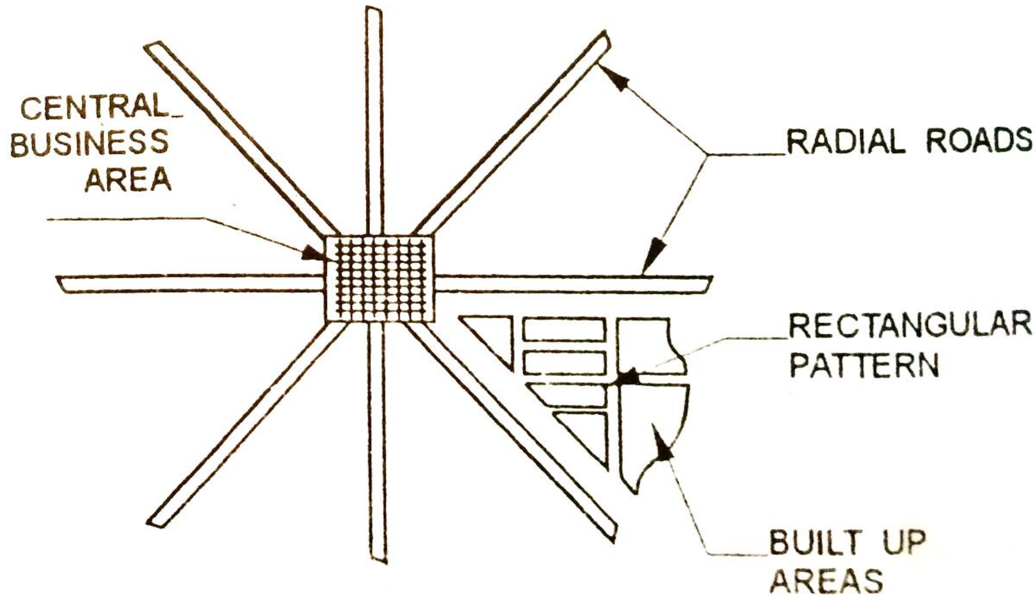


Fig 4.1(b) Radial or star and block pattern

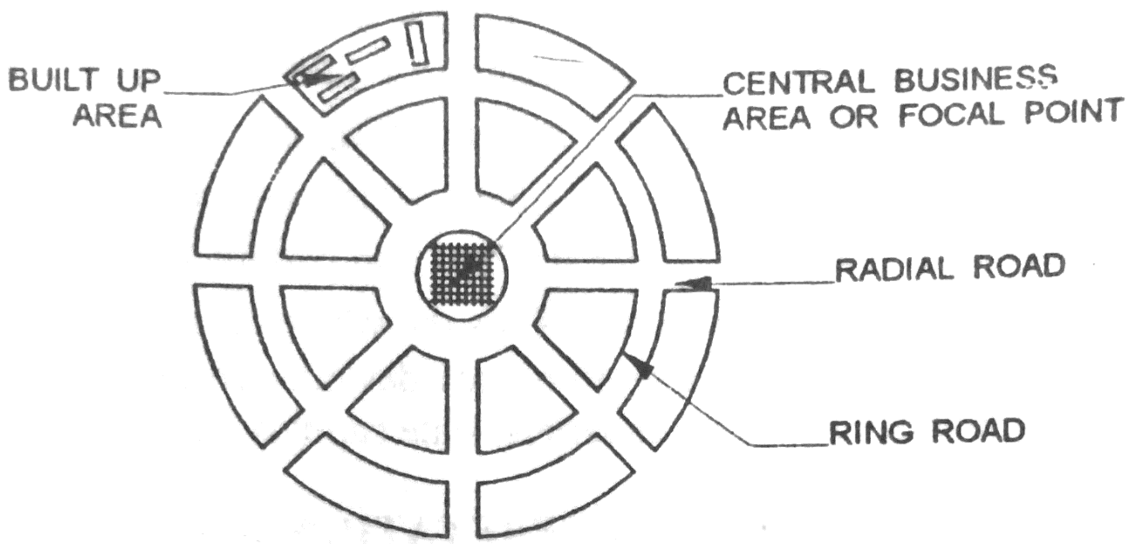
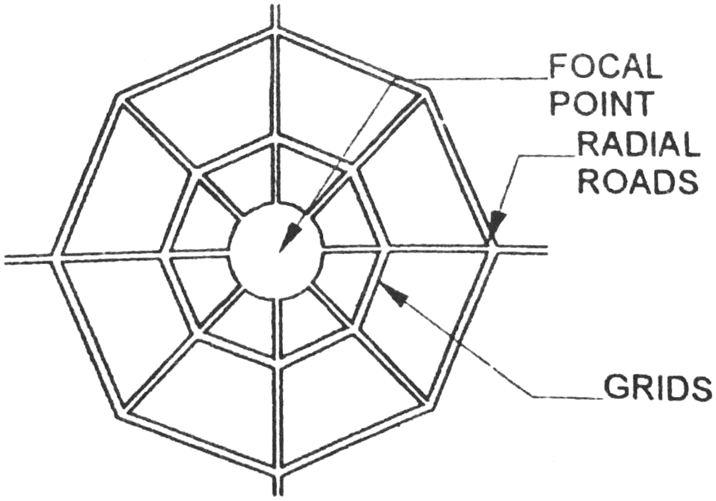


Fig. 4.1 (c) Radial or star and circular pattern



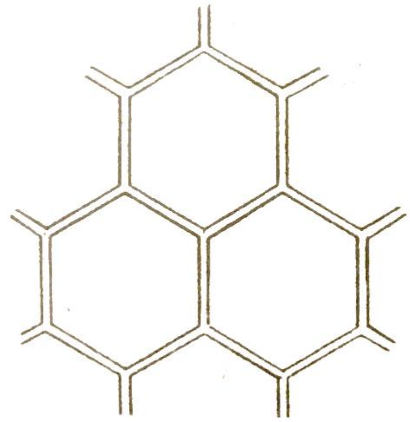
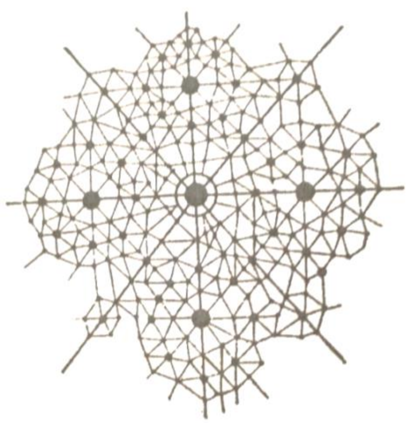


Fig. 4.1 (d) Radial or star and grid pattern

Fig. 4.1 (e) Hexagonal pattern



Legend : City centre – encircled dot, sector centers - ●, Suburban centers - •
Neighbourhood centers - •, Representation of a "Minimum Travel" city
(Assumed population of 2 million)

Fig. 1.1 (f) Minimum travel Pattern

Each of these patterns has its own advantages and limitations. There can be a number of other geometric patterns also. The choice of the pattern very much depends on the locality, the layout of the different towns, villages, industrial and production centers and on the choice of the planning engineer.

The rectangular or the block pattern has been adopted in the city of Chandigarh. But from traffic operation point this is not consider convenient. An example of radial and circular pattern is the road network of Connaught place in New Delhi. The Nagpur road plan formulae were prepared assuming “Star and Grid pattern”.

The concept of star and grid patterns has been explained below are illustrated in the figures.

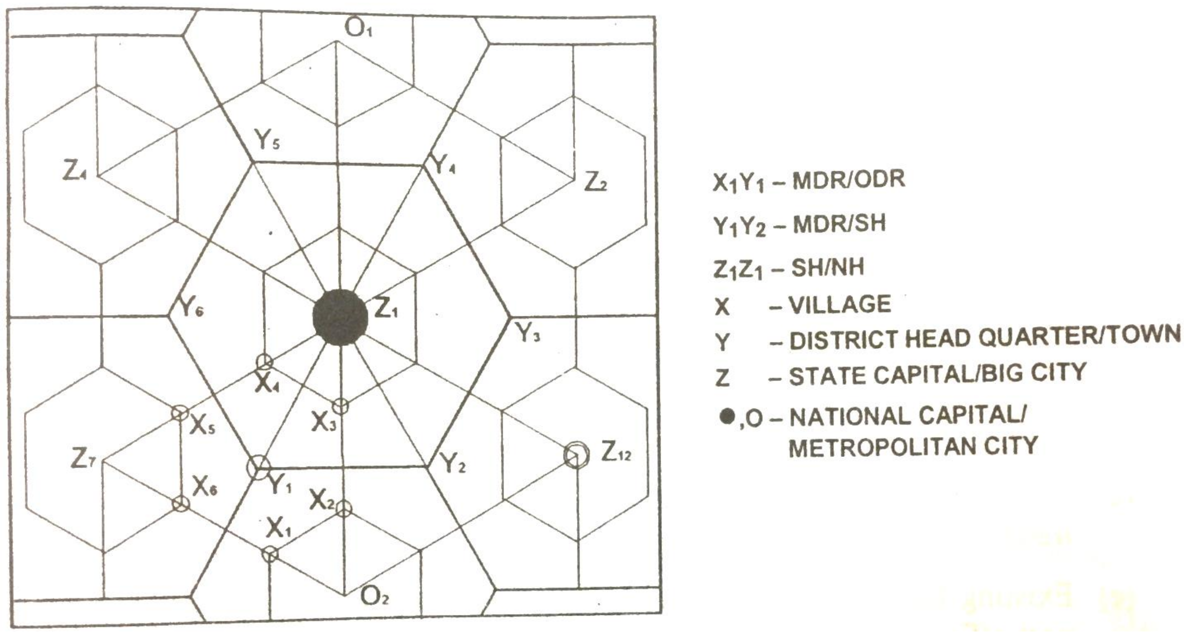


Fig. 1.1(g) concept of star and Grid

In the star and g o be villages, points Y the towns and points Z repre Points Y1, Y2, Y3 etc. act as focal points for connecting the villages X1,X2,X3 etc. similarly Z1,Z2,Z3 etc. are focal points connecting the town Y1,Y2,Y3. Thus star and grid pattern is formed between X1,X2,X3 etc. similarly bigger star and grid pattern are formed with Y1,Y2,Y3 etc. and Z1,Z2,Z3 etc. as focal points. Thus the whole area can thus be covered on an expanding scale. Such a network therefore, provides inter-communication facilities to each of the villages, towns, district headquarters, state capitals etc.

6. Highway Alignment

The position or layout of centre line of the highway on the ground is called the alignment. It includes straight path, horizontal deviation and curves. Due to improper alignment, the disadvantages are,

i. Increase in construction

ii. Increase in maintenance cost

iii. Increase in vehicle operation cost

iv. Increase in accident cost

Once the road is aligned and constructed, it is not easy to change the alignment due to increase in cost of adjoining land and construction of costly structure.

6.1 Requirements of highway alignment

i. Short

ii. Easy

iii. Safe and

iv. Economical

i. **Short** –desirable to have a short alignment between two terminal stations.

ii. **Easy**-easy to construct and maintain the road with minimum problem also easy for operation of vehicle.

iii. **Safe**-safe enough for construction and maintenance from the view point of stability of natural hill slope, embankment and cut slope also safe for traffic operation.

iv. **Economical**-total cost including initial cost, maintenance cost and vehicle operation cost should be minimum.

6.2 Factors controlling alignment

i. Obligatory points

ii. Traffic

iii. Geometric design

iv. Economics

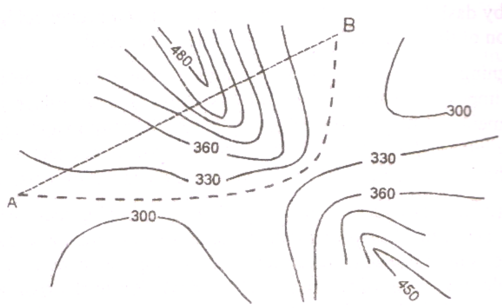
v. Other considerations

6.2.1 Obligatory Points

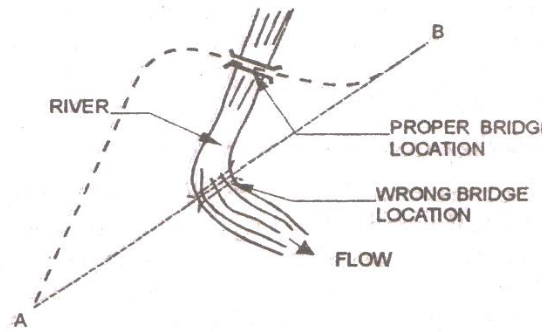
These are control points governing the alignment of the highways. These control points may be divided broadly into two categories.

(i) Points through which the alignment is to pass

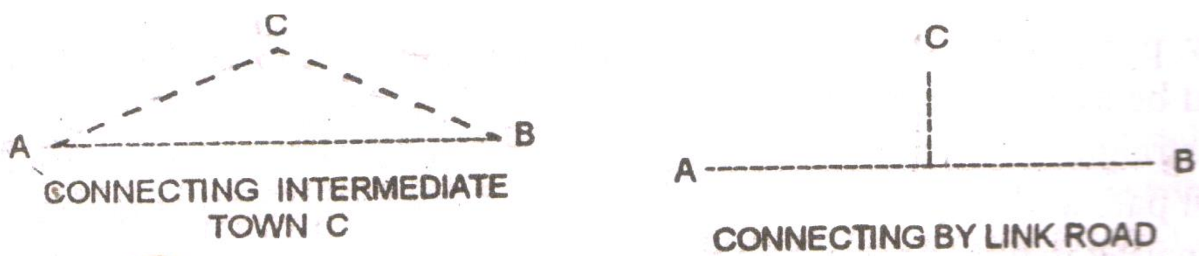
(ii) Points through which the alignment should not pass



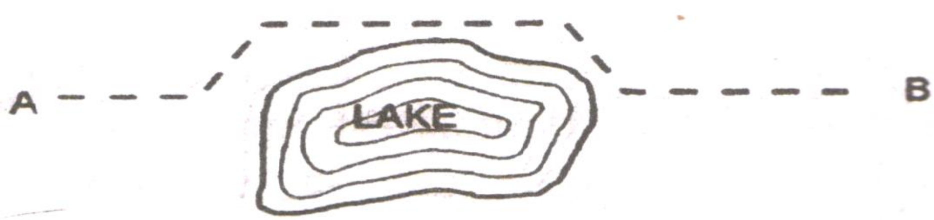
(a) Alignment along hill side pass



(b) Alignment to suit proper bridge location



(c) Alignment to connect intermediate area



(d) Alignment avoiding intermediate area

5.2.1 Obligatory points Controlling Alignment of Road

i) Points through which the alignment has to pass

Obligatory points through which the road alignment has to pass are generally due to the topographic and other site conditions including natural obstructions. Some of the examples of this category include location of a mountain pass, suitable location of bridge to cross a river, presence of quarry or an intermediate town to be connected. These obligatory points necessitate deviation of the road alignment from the straight alignment with shortest or easiest path.

When the road alignment has to cross hill range, mountains or high ridges, different alternatives to be considered are; (a) to cut a tunnel across the hill or mountain (b) to go round the hill (c) to deviate until a suitable mountain pass is available. The choice or suitability of these alternatives depend on many other factors, like the topography, site conditions and cost considerations. Fig 1.3(a) shows how the straight alignment AB is deviated along the hill side pass, thus avoiding a tunnel or heavy cutting.

The road bridge across a river can be located only at a place where the river has straight and permanent path and not where there is a bend in the river; also the selected location of the bridge should be such that the abutment and pier can be properly constructed. The road approaches to this bridge should not be curved near the bridge and as far as possible skew crossing of the river should be avoided. Thus in order to locate a bridge across a river, the alignment may have to be changed. Fig 1.3(b) shows that the straight alignment between stations A and B which crosses the river at the bend is not a suitable location and hence the alignment is to be deviated along the path shown (by dash lines) in order to cross the river at a proper bridge location at the straight portion of the river on the up-stream side of the bend.

While aligning a road between two stations, it may often be desirable to connect some of the important intermediate towns, villages or other place of interest. The straight alignment AB may be shifted along line ACB, as shown in Fig. 1.3(c) in order to connect the intermediate station C. It is also possible to connect the station C with a link road as shown in this figure, thus avoiding the deviation of the straight alignment.

Fig 1.3 (d) illustrates an instance when the straight alignment AB is encountered with a lake in between. It is possible to consider two different alternatives to take the road project forward namely, (1) construction of a long bridge across the lake along the original straight alignment of the road and (2) deviation of the road alignment and to take the road around the lake. Proposal to construct a long bridge across the lake along the straight alignment as per alternative (1) will be very expensive and time consuming apart from the additional cost for under-water construction of the bridge sub-structure. Taking the road alignment around the lake as per alternative (2) will increase the road length and consequently the road user cost will also be higher. But the total project cost of this alternative is likely to be lower than the alternative (1), construction of the long bridge structure across the lake.

ii) Points through which the alignment should not pass

Religious places: These have been protected by the law from being acquired for any purpose. Therefore, these points should be avoided while aligning.

Very costly structures: Acquiring such structures means heavy compensation which would result in an increase in initial cost. So the alignment may be deviated not to pass through that point.

Lakes/Ponds etc: The presence of a lake or pond on the alignment path would also necessitate deviation of the alignment.

6.2.2 Traffic

Alignment should be selected based on traffic surveys. Origin and Destination study should be carried out in that area and also we have to consider the future development in that road network.

6.2.3 Geometric Design

Alignment is decided based on the design of horizontal and vertical curves, sight distance and gradient of that section. It is also decided based on the Design Speed of that Highway.

6.2.4 Economy

It is based on the initial cost of construction and maintenance cost of the road, if it a shortest path the cost of construction will be reduced.(Decision is based on Quantity of Cutting and Filling of Earth.)

Special considerations or care for Hill Roads

Common problems in hill roads are land sliding, stability of road, providing adequate drainage facility, reducing hairpin bends, needless raise and fall.

Additional care in hill roads

- Stability
- Drainage

- Geometric standards of hill roads
- Resisting length

7. Engineering Surveys for Highway locations

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

i. Map study (Provisional alignment Identification)
ii. Reconnaissance survey
iii. Preliminary survey
iv. Final location and detailed surveys

7.1 Map Study

- From the map alternative routes can be suggested in the office, if the topographic map of that area is available.
- The probable alignment can be located on the map from the fallowing details available on the map.

a) Avoiding valleys, ponds or lake
b) Avoiding bend of river
c) If road has to cross a row of hills, possibility of crossing through mountain pass.
d) When road is connected between 2 stations one at mountain top and other at foot alternate route be suggested based on gradient.

- Map study gives a rough guidance of the routes to be further surveyed in the field

7.2 Reconnaissance Survey

To examine the general characters of the area confirm features indicated on map.

- To examine the general character of the area infield for deciding the most feasible routes for detailed studies.
- A survey party may inspect along the proposed alternative routes of the map in the field with very simple instruments like abney level, tangent clinometers, barometer, etc. To collect additional details.

Details to be collected from alternative routes during this survey are:

- Valleys, ponds, lakes, marshy land, hill, permanent structures and other obstruction.
- Value of gradient, length of gradient and radius of curve.
- Number and type of cross drainage structures.
- High Flood Level(HFL)
- Soil Characteristics.
- Geological features.
- Source of construction materials – stone quarries, water sources.

Prepare a report on merits and demerits of different alternative routes.

As a result a few alternate alignments maybe chosen for further study based on practical considerations observed at the site.

7.3 Preliminary survey

Objective of preliminary survey are:

- To survey the various alternative alignments proposed after the reconnaissance and to collect all the necessary physical information and detail of topography, drainage and soil.
- To compare the different proposals in view of the requirements of the good alignment.

- To estimate quantity of earthwork, material and construction aspects to workout various alternatives.
- The alignment finalized at the design office after the preliminary survey.

7.4 Final location and detailed survey

- Be first located on the field by establishing the centerline.

7.4.1 Location survey:

- Transferring the alignment onto ground. This is done by transit theodolite.
- Major and minor control points are established on the ground and centre pegs are driven, checking the geometric design requirements.
- Centerline stacks are driven at suitable intervals, say 50m interval in plane and rolling terrains and 20min hilly terrain.

7.4.2 Detailed survey:

- Temporary bench marks are fixed at intervals of about 250m and at all drainage and under pass structure.
- Earthwork calculations and drainage details are to be workout from the level books.
- Cross-sectional levels are taken at intervals of

  50 – 100 m Plane terrain,

  50-75m Rolling terrain,

  50 m built – up area,

  20 m Hilly areas.
- Topographical details are noted down

- Detail soil survey is to be carried out.
- CBR value of the soils along the alignment may be determined for design of pavement.
- Thedataduringdetailedsurveyshouldbeelaborateandcompleteforpreparingdetailedplans, design and estimates of project.

TRANSPORTATION ENGINEERING

UNIT-III

Objective:

To acquire design principles of Highway geometrics and pavements

Syllabus: HIGHWAY GEOMETRIC DESIGN

Important of Geometric Design, Design Controls and Criteria, Highway Cross Section Elements, Sight Distance Elements, Stopping Sight Distance, Overtaking Sight Distance and Intermediate Sight Distance, Design of Horizontal Alignment, curves, Transition Curves – Objectives of providing transition curves, different types of transition curves, Calculation of length of transition curve, Vertical Alignment- Gradients – categories of gradients – compensation in gradient on horizontal curves, Vertical Curves – different types of vertical curves.

Learning Outcomes:

After completion of this unit the student will be able to

- Explain highway cross section elements.
- Calculate the stopping sight distance and overtaking sight distance.
- Calculate the super elevation and extra widening in curve.
- Explain types of transition curve.
- Calculate the length of vertical curve

Learning Material

2.1 Importance of geometric Design

The geometric design of highways deals with the dimensions and layout of visible features of the highway. The emphasis of the geometric design is to address the requirement of the driver and the vehicle such as safety, comfort, efficiency, etc.

Geometric design of highways deals with following elements:

i. Cross section elements
ii. Sight distance consideration
iii. Horizontal alignment details
iv. Vertical alignment details
v. Intersection elements

2.1.1 Factors affecting geometric design

Factors affecting the geometric designs are as follows:

i. Design speed
ii. Topography
iii. Traffic factors
iv. Design hourly volume and capacity
v. Environmental and other factors

Design speed: Design speed is the single most important factor that affects the geometric design. It directly affects the sight distance, horizontal curve, and the length of vertical curves. Since the speed of vehicles vary with driver, terrain etc, a design speed is adopted for all the geometric design.

Topography: It is easier to construct roads with required standards for a plain terrain. However, for a given design speed, the construction cost increases multi form with the gradient and the terrain.

Traffic: It will be uneconomical to design the road for peak traffic flow. Therefore a reasonable value of traffic volume is selected as the design hourly volume which is determined from the various traffic data collected.

Environmental: Factors like air pollution, noise pollution etc. should be given due consideration in the geometric design of roads.

Economy: The design adopted should be economical as far as possible. It should match with the funds allotted for capital cost and maintenance cost.

Others: Geometric design should be such that the aesthetics of the region is not affected

2.2 Cross sectional elements

The feature of the cross-section of the pavement influences the life of the pavement as well as the riding comfort and safety.

2.2.1 Pavement surface characteristics:

For a safe and comfortable driving four aspects of the pavement surface are important;

Friction: Friction between the wheel and the pavement surface is a crucial factor in the design of horizontal curves and thus the safe operating speed. Further, it also affects the acceleration and deceleration ability of vehicles. Lack of adequate friction can cause skidding or slipping of vehicles. Various factors that affect friction are: Type of the pavement (like bituminous, concrete, or gravel), Condition of the pavement (dry or wet, hot or cold, etc.), Condition of the tire (new or old), and Speed and load of the vehicle. The choice of the value of f is a very complicated issue since it depends on many variables. IRC suggests the coefficient of longitudinal friction as 0.35-0.4 depending on the speed and coefficient of later friction as 0.15.

Unevenness: It affects the vehicle operating cost, speed, riding comfort, safety, fuel consumption and wear and tear of tires. Unevenness index is a measure of unevenness which is the cumulative measure of vertical undulation of the pavement surface recorded per unit horizontal length of the road.

Light reaction: White roads have good visibility at night, but caused glare during day time. Black roads has no glare during day, but has poor visibility at night

Drainage: The pavement surface should be absolutely impermeable to prevent seepage of water into the pavement layers.

2.2.2 Cross Slope or Camber

Camber or cant is the cross slope provided to raise middle of the road surface in the transverse direction to drain off rain water from road surface. The objectives of providing camber are: Surface protection especially for gravel and bituminous roads Sub-grade protection by proper drainage. Quick drying of pavement which in turn increases safety.

Too steep slope is undesirable for it will erode the surface. Camber is measured in 1 in n or n% (Eg. 1 in 50 or 2%) and the value depends on the type of pavement surface. The values suggested by IRC for various categories of pavement is given in Table 2.1. The common types of camber are parabolic, straight, or combination of them (Fig 2.1)

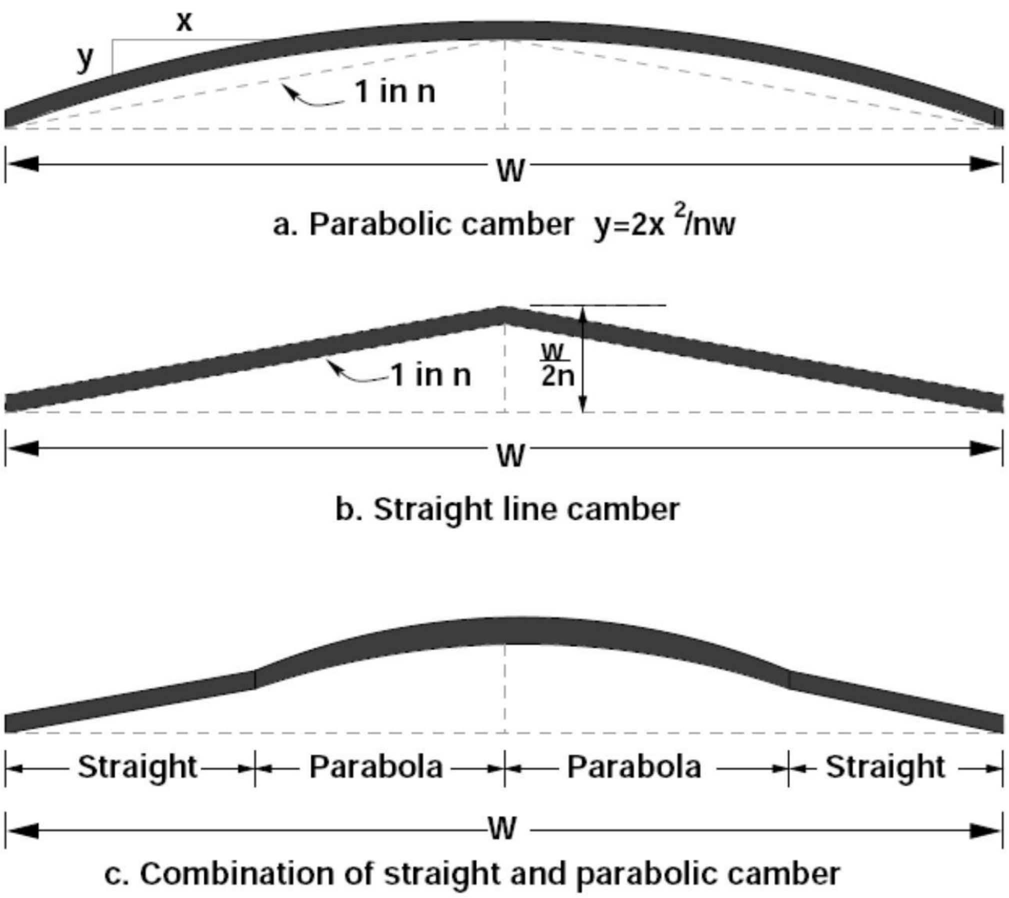


Fig. 2.1 Different types of camber

Table 2.1: IRC Values for camber

Type of surface	Heavy Rainfall	Low Rainfall
CC roads	1 in 50	1 in 60
Thin bituminous roads	1 in 40	1 in 50
Water Bound Macadam roads	1 in 33	1 in 40
Village Roads	1 in 25	1 in 33

2.2.3 Width of pavement or Carriageway

Width of the carriage way or the width of the pavement depends on the width of the traffic lane and number of lanes. Width of a traffic lane depends on the width of the vehicle and the clearance. Side clearance improves operating speed and safety. The maximum permissible width of a vehicle is 2.44 and the desirable side clearance for single lane traffic is 0.68 m. This require minimum of lane width of 3.75 m for a single lane road (Fig 2.2). However, the side clearance required is about 0.53 m, on both side and 1.06 m in the center. Therefore, a two lane road require minimum of 3.5 meter for each lane (Fig 2.2). The desirable carriage way width recommended by IRC is given in Table 2.2.

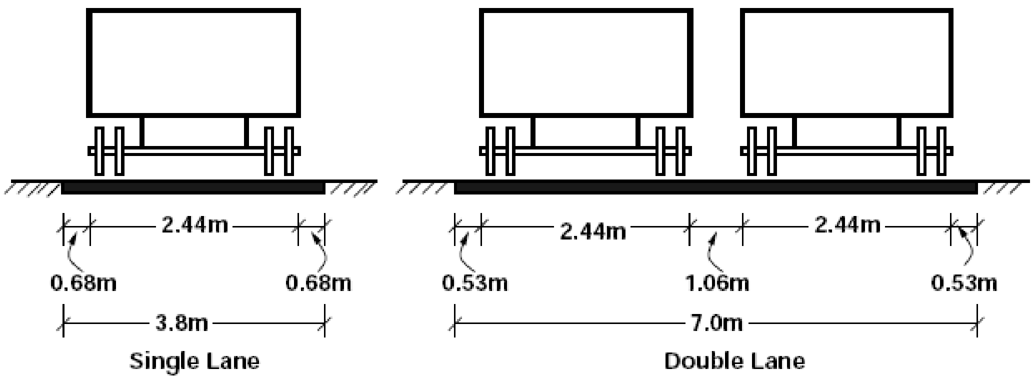


Fig. 2.2 Lane width for single and two lane roads

Table 22: IRC Speciation for carriage way width

Class of road	Width of carriageway
Single lane	3.75 m
Two lane, no kerbs	7.0 m
Two lane, raised kerbs	7.5 m
Intermediate carriage	5.5 m
Multi-lane	3.5 m per lane

2.2.4 Kerbs

Kerbs indicate the boundary between the carriage way and the shoulder or islands or footpaths. Different types of kerbs are (Fig. 2.3).

Low or mountable kerbs: These types of kerbs are provided such that they encourage the traffic to remain in the through traffic lanes and also allow the driver to enter the shoulder area with little difficulty.

Semi-barrier type kerbs: When the pedestrian traffic is high, these kerbs are provided. Their height is 15 cm above the pavement edge.

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

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

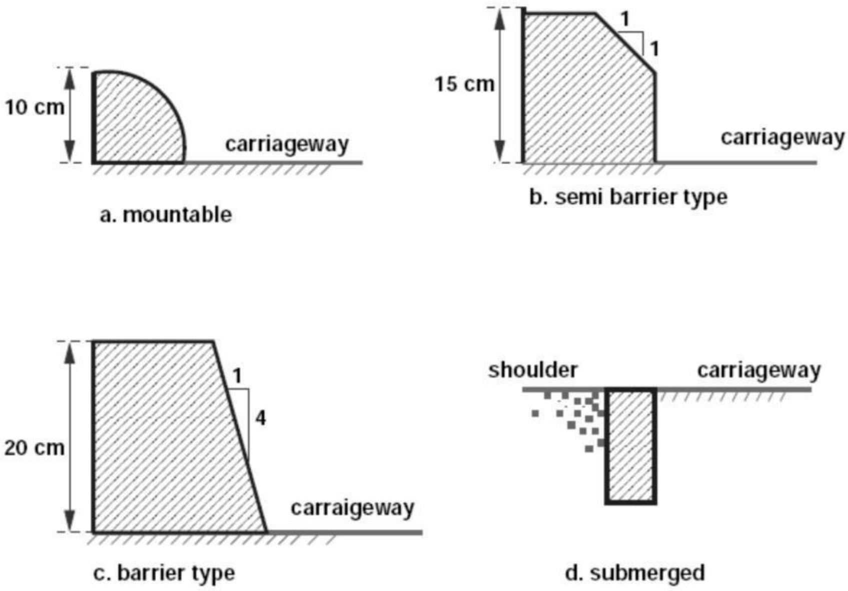


Fig. 2.3 Different types of Kerbs.

2.2.5 Road margins

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

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

Parking lanes: Parking lanes are provided in urban lanes for side parking. Parallel parking is preferred because it is safe for the vehicles moving in the road. The parking lane should have a minimum of 3.0 m width in the case of parallel parking.

Bus-bays: Bus bays are provided by recessing the kerbs for bus stops. They are provided so that they do not obstruct the movement of vehicles in the carriage way.

Service roads: Service roads or frontage roads give access to access controlled highways like freeways and expressways. They run parallel to the highway and will be usually isolated by a separator and access to the highway will be provided only at selected points.

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

Footpath: Footpaths are exclusive right of way to pedestrians, especially in urban areas. They are provided for the safety of the pedestrians when both the pedestrian traffic and vehicular traffic is high. **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 on the embankment, especially when the height of the fill exceeds 3 m.

2.2.6 Width of formation

Width of formation or roadway width is the sum of the widths of pavements or carriage way including separators and shoulders. This does not include the extra land in formation/cutting. The values suggested by IRC are given in Table 2.3.

Table 2.3 Width of formation for various classed of roads

Road classification	**Roadway width in m**	
	Plain and Rolling terrain	**Mountainous and Steep terrain**
NH / SH	12.00	6.25 – 8.80
MDR	9.00	4.75
ODR	7.50 – 9.00	4.75
VR	7.50	4.00

2.2.7 Right of way

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

Width of formation**:** It depends on the category of the highway and width of roadway and road margins. Height of embankment or depth of cutting: It is governed by the topography and the vertical alignment. Side slopes of embankment or cutting: It depends on the height of the slope, soil type etc. Drainage system and their size which depends on rainfall, topography etc. The importance of reserved land is emphasized by the following Extra width of land is available for the construction of roadside facilities.

The importance of reserved land is emphasized by the following. Extra width of land is available for the construction of roadside facilities. Land acquisition is not possible later, because the land may be occupied for various other purposes (buildings, business etc.) The normal ROW requirements for built up and open areas as specified by IRC is given in Table 2.4. A typical cross section of a ROW is given in Fig.2.4.

Table 2.4 Normal right of way for open areas

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

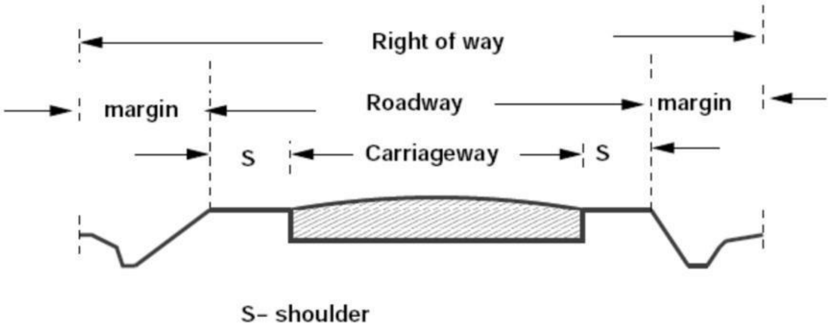


Fig. 2.4 A typical Right of way (ROW)

2.3 Sight distance

The safe and efficient operation of vehicles on the road depends very much on the visibility of the road ahead of the driver.

2.3.1 Types of sight distance

Sight distance available from a point is the actual distance along the road surface, over which a driver from a specified height above the carriage way has visibility of stationary or moving objects.

Three sight distance situations are considered for design:

1. Stopping sight distance (SSD) or the absolute minimum sight distance
2. Intermediate sight distance (ISD) is the defined as twice SSD and
3. Overtaking sight distance (OSD) for safe overtaking operation
4. Head light sight distance is the distance visible to a driver during night driving under the illumination of head light Safe sight distance to enter into an intersection distance depends on:

2.3.2 Stopping sight distance (SSD)

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

The computation of sight distance depends on:

- Reaction time of the driver:

  Reaction time of a driver is the time taken from the instant the object is visible to the driver to the instant when the brakes are applied. The total reaction time may be split up into four components based on PIEV theory. In practice, all these times are usually combined into a total perception- reaction time suitable for design purposes as well as for easy measurement.
- Speed of the vehicle:

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

- Efficiency of brakes:
  The efficiency of the brakes depends upon the age of the vehicle, vehicle characteristics etc. If the brake efficiency is 100%, the vehicle will stop the moment the brakes are applied. But practically, it is not possible to achieve 100% brake efficiency.
- Frictional resistance between the tire and the road:
  The frictional resistance between the tire and road plays an important role to bring the vehicle to stop. When the frictional resistance is more, the vehicles stop immediately. Thus sight required will be less. No separate provision for brake efficiency is provided while computing the sight distance.
- Gradient of the road:
  Gradient of the road also affects the sight distance. While climbing up a gradient, the vehicle can stop immediately. Therefore sight distance required is less.

2.3.2.1 Analysis of stopping distance

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

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.

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

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

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

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

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,

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

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 α = W tanα = Wn=100. Equating kinetic energy and work done:

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

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

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

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

2.3.3 Overtaking sight distance (OSD)

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

The factors that affect the OSD are:

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

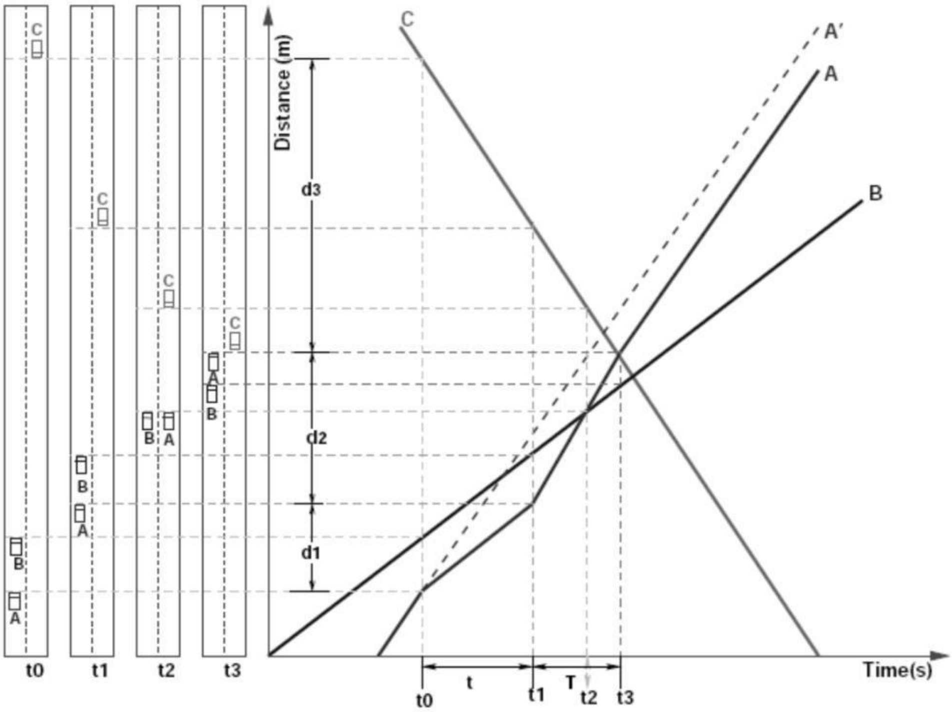


Fig. 2.5 Time-space diagram: Illustration of overtaking sight distance

The dynamics of the overtaking operation is given in the Fig which is a time-space diagram. The x-axis denotes the time and y-axis shows the distance travelled by the vehicles. The trajectory of the slow moving vehicle (B) is shown as a straight line which indicates that it is traveling at a constant speed. A fast moving vehicle (A) is traveling behind the vehicle B. The trajectory of the vehicle is shown initially with a steeper slope. The dotted line indicates the path of the vehicle A if B was absent. The vehicle A slows down to follow the vehicle B as shown in the Fig with same slope from t_0 to t_1. Then it overtakes the vehicle B and occupies the left lane at time t_3. The time duration $T = t_3 - t_1$ is the actual duration of the overtaking operation. The snapshots of the road at time $t_{0,}$ t_1, and t_3 are shown on the left side of the Fig. From the Fig.2.5, the overtaking sight distance consists of three parts.

d_1 the distance travelled by overtaking vehicle A during the reaction time $t = t_1 - t_0$

d_2 the distance travelled by the vehicle during the actual overtaking operation $T = t_3 - t_1$

d_3 is the distance travelled by on-coming vehicle C during the overtaking operation (T).

Therefore:

$$OSD = d_1 + d_2 + d_3$$

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

$$d_1 = v_b t$$

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

$$s = 0.7v_b + 6$$

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

$$d_2 = v_bT+\frac{1}{2}\alpha T^2$$

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

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

$$d_3 = vT$$

The overtaking sight distance is (Fig.2.5)

$$OSD = d_1+d_2+d_3$$

Where v_b is the velocity of the slow moving vehicle in m/s^2, t is the reaction time of the driver in sec, s is the spacing between the two vehicle in m given by equation and a is the overtaking vehicles acceleration in m/s^2. In case the speed of the overtaken vehicle is not given, it can be assumed that it moves 16 kmph slower the design speed.

The acceleration values of the fast vehicle depends on its speed and given in Table 2.5

Table 2.5: Maximum overtaking acceleration at different speeds

Speed (kmph)	**Maximum overtaking acceleration (m/sec^2)**
25	1.41
30	1.30
40	1.24
50	1.11

65	0.92
80	0.72
100	0.53

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

2.3.3.1 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 (Fig. 2.6).

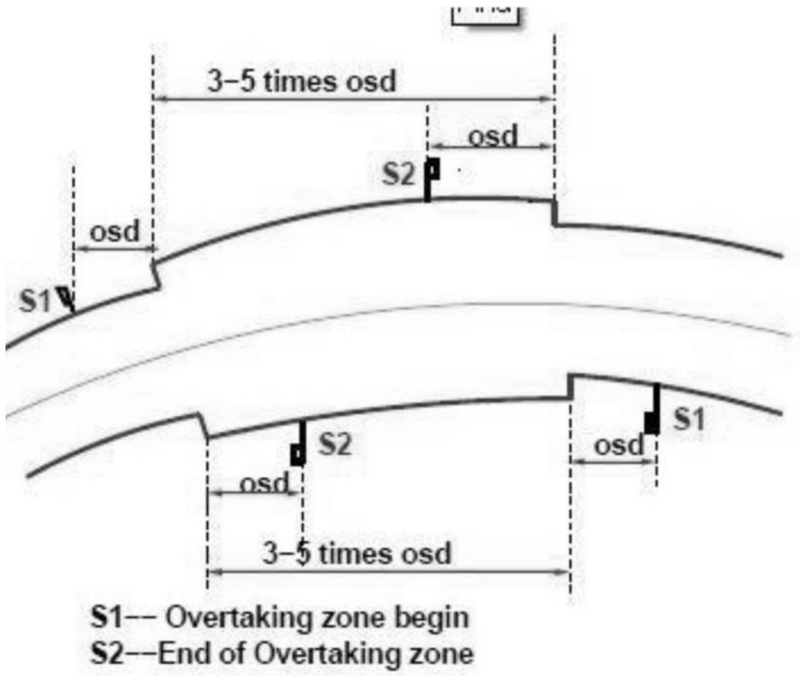


Fig. 2.6 Overtaking zones

2.3.3.2 Sight distance at intersections

At intersections where two or more roads meet, visibility should be provided for the drivers approaching the intersection from either sides. They should be able to perceive a hazard and stop the vehicle if required. Stopping sight distance for each road can be computed from the design speed. The sight distance should be provided such that the drivers on either side should be able to see each other. This is illustrated in the Fig. 2.7.

Design of sight distance at intersections may be used on three possible conditions:

- Enabling approaching vehicle to change the speed
- Enabling approaching vehicle to stop

* Enabling stopped vehicle to cross a main road

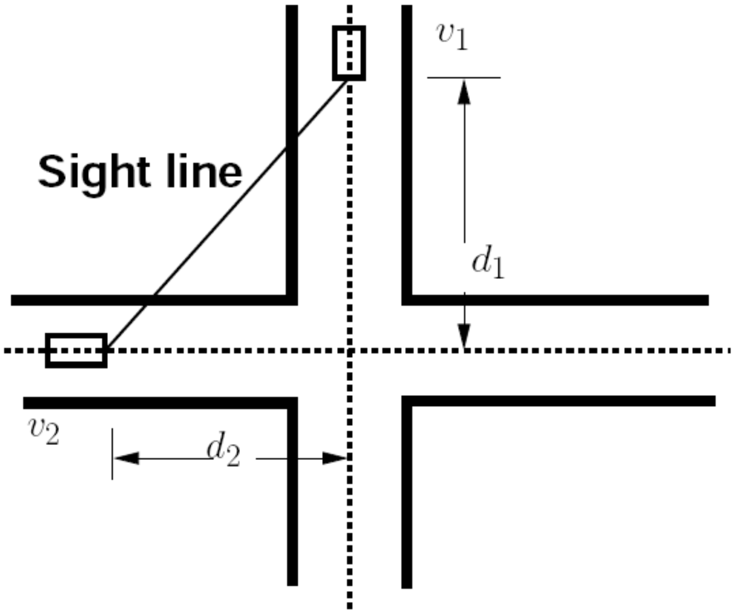


Fig. 2.7 Sight distance at intersections

2.4 Horizontal Alignment

Horizontal alignment is one of the most important features influencing the efficiency and safety of a highway. A poor design will result in lower speeds and resultant reduction in highway performance in terms of safety and comfort. In addition, it may increase the cost of vehicle operations and lower the highway capacity. Horizontal alignment design involves the understanding on the design aspects such as design speed and the effect of horizontal curve on the vehicles. The horizontal curve design elements include design of super elevation, extra widening at horizontal curves, design of transition curve, and set back distance.

2.4.1 Design Speed

The design speed, as noted earlier, is the single most important factor in the design of horizontal alignment. The design speed also depends on the type of the road. For e.g, the design speed expected from a National highway will be much higher than a village road, and hence the curve geometry will vary significantly.

The design speed also depends on the type of terrain. A plain terrain can afford to have any geometry, but for the same standard in a hilly terrain requires substantial cutting and filling implying exorbitant costs as well as safety concern due to unstable slopes. Therefore, the design speed is normally reduced for terrains with steep slopes.

For instance, Indian Road Congress (IRC) has classified the terrains into four categories, namely plain, rolling, mountainous, and steep based on the cross slope as given in Table 2.6. Based on the type of road and type of terrain the design speed varies. The IRC has suggested desirable or ruling speed as well as minimum suggested design speed and is tabulated in table 14:2. The recommended design speed is given in Table 2.7.

Table 2.6 Terrain classification

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

Table 2.7 Design speed in kmph as per IRC (ruling and minimum)

Type	plain	Rolling	Mountainous	Steep
NH & SH	100 – 80	80 – 65	50 – 40	40 – 30
MDR	80 – 65	65 – 50	40 – 30	30 – 20
ORD	65 – 50	50 – 40	30 – 25	25 – 20
VR	50 - 40	40 - 35	25 - 20	25 – 20

2.4.2 Horizontal curve

The presence of horizontal curve imparts centrifugal force which is reactive force acting outward on a vehicle negotiating it. Centrifugal force depends on speed and radius of the horizontal curve and is counteracted to a certain extent by transverse friction between the tyre and pavement surface. On a curved road, this force tends to cause the vehicle to overrun or to slide outward from the centre of road curvature. For proper design of the curve, an understanding of the forces acting on a vehicle taking a horizontal curve is necessary. Various forces acting on the vehicle are illustrated in the Fig.2.8.

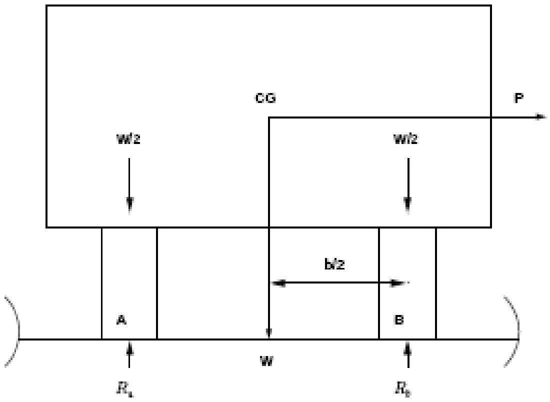


Fig. 2.8 Effect of horizontal curve

They are the centrifugal force (P) acting outward, weight of the vehicle (W) acting downward, and the reaction of the ground on the wheels (R_A and R_B). The centrifugal force and the weight is assumed to be from the centre of gravity which is at h units above the ground. Let the wheel base be assumed as b units. The centrifugal force P in kg/m^2 is given by

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

where W is the weight of the vehicle in kg, v is the speed of the vehicle in m/sec, g is the acceleration due to gravity in m/sec^2 and R is the radius of the curve in m.

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

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

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

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

At the equilibrium over turning is possible when

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

and for safety the following condition must satisfy:

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

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 tire. The transverse skid resistance (F) is given by

$$F = F_A + F_B$$

$$= f\,(R_A + R_B)$$
$$= f\,W$$

where F_A and F_B is the fractional force at tire A and B, R_A and R_B is the reaction at tire 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

At equilibrium, when skidding takes place

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

and for safety the following condition must satisfy:

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

2.4.1 Analysis of super-elevation

Super-elevation or cant or banking is the transverse slope provided at horizontal curve to counteract the centrifugal force, by raising the outer edge of the pavement with respect to the inner edge, throughout the length of the horizontal curve. When the outer edge is raised, a component of the curve weight will be complimented in counteracting the effect of centrifugal force. In order to find out how much this raising should be, the following analysis may be done. The forces acting on a vehicle while taking a horizontal curve with superelevation is shown in Fig.2.9.

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

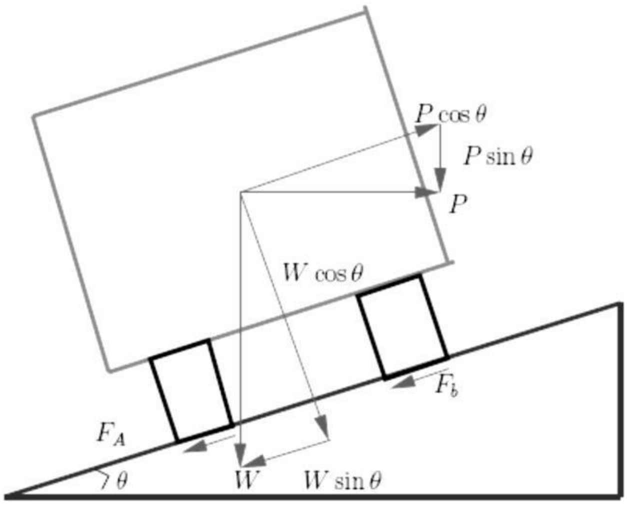


Fig. 2.9 Analysis of superelevation

- P the centrifugal force acting horizontally out-wards through the center of gravity,
- W the weight of the vehicle acting down-wards through the center of gravity, and
- F the friction force between the wheels and the pavement, along the surface inward. At equilibrium, by resolving the forces parallel to the surface of the pavement we get,

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

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

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

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

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

We have already derived as expression for P/W.

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

Relatively f $\tan\theta$ can be neglected.

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

2.4.1 Radius of Horizontal Curve

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

$$R_{ruling} = \frac{v^2}{ge}$$

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

2.4.2 Extra Widening

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

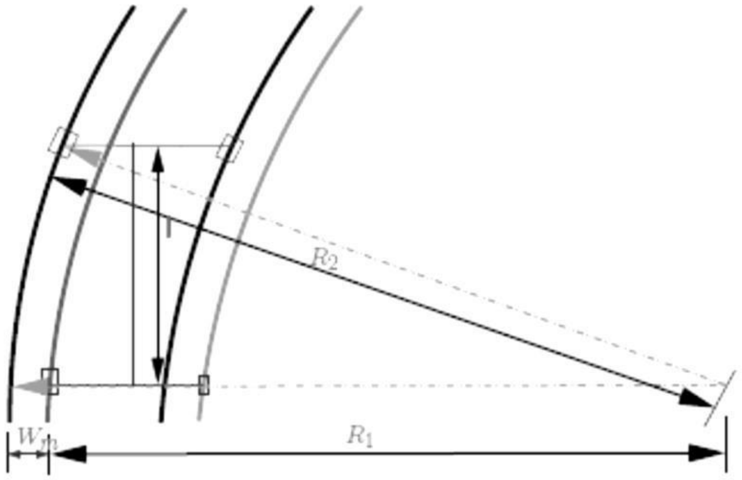


Fig. 2.10 Extra-widening at a horizontal curve

2.4.5.1 Mechanical widening

The reasons for the mechanical widening are: When a vehicle negotiates a horizontal curve, the rear wheels follow a path of shorter radius than the front wheels as shown in Fig. 2.10. This phenomenon is called off-tracking, and has the effect of increasing the effective width of a road space required by the vehicle. Therefore, to provide the same clearance between vehicles traveling in opposite direction on curved roads as is provided on straight sections, there must be extra width of carriageway available. This is an important factor when high proportion of vehicles are using the road. Trailor trucks also need extra carriageway, depending on the type of joint. In addition speeds higher than the design speed causes transverse skidding which requires additional width for safety purpose. The expression for extra width can be derived from the simple geometry of a vehicle at a horizontal curve as shown in Fig. 2.10. 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 given below

Please note that for large radius, $R_2 \approx R$, which is the mean radius of the curve, then W_m is given by:

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

Psychological widening

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

$W_e =$

2.4.6 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.

2.4.6.1 Type of transition curve

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

1. It full fills 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 1 at the straight edge and changes to R at the curve point ($L_s \alpha 1/R$) and calculation and field implementation is very easy.

2.4.6.2 Length of transition curve:

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

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:

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

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

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

$$C = \frac{80}{75+V} \text{ } (C_{min} = 0.5, C_{max} = 0.8)$$

2. Rate of introduction of super-elevation

Raise (E) of the outer edge with respect to inner edge is given by E = eB = e(W +We). The rate of change of this raise from 0 to E is achieved gradually with a gradient of 1 in N over the length of the transition curve (typical range of N is 60-150). Therefore, the length of the transition curve Ls2 is:

$$L_{s2} = N_e \, (W + W_e)$$

3. By empirical formula IRC suggest the length of the transition curve is minimum for a plain and rolling terrain:

$$L_{s3} = \frac{2.7v^2}{R}$$

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

2.5. Vertical Alignment

The vertical alignment of a road consists of gradients (straight lines in a vertical plane) and vertical curves. The vertical alignment is usually drawn as a profile, which is a graph with elevation as vertical axis and the horizontal distance along the centre line of the road as the the horizontal axis.

2.5.1 Gradient

Gradient is the rate of rise or fall along the length of the road with respect to the horizontal. While aligning a highway, the gradient is decided designing the vertical curve.Before finalising the gradients, the construction cost, vehicular operation cost and the practical problems in the site also has to be considered.

2.5.1.1 Types of gradient

Many studies have shown that gradient upto seven percent can have considerable effect on the speeds of the passenger cars. On the contrary, the speeds of the heavy vehicles are considerably reduced when long gradients a sat as two percent is adopted. Although, atter gradients are desirable, it is evident that the cost of construction will also be very high.

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

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.

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.

Exceptional gradient Exceptional gradient are very steeper gradients given at unavoidable situations. They should be limited for short stretches not exceeding about 100 meters at a stretch.

2.5.2 Summit curve

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

1. When a positive gradient meets another positive gradient

2. When positive gradient meets a at gradient

3. When an ascending gradient meets a descending gradient.

4. When a descending gradient meets another descending gradient

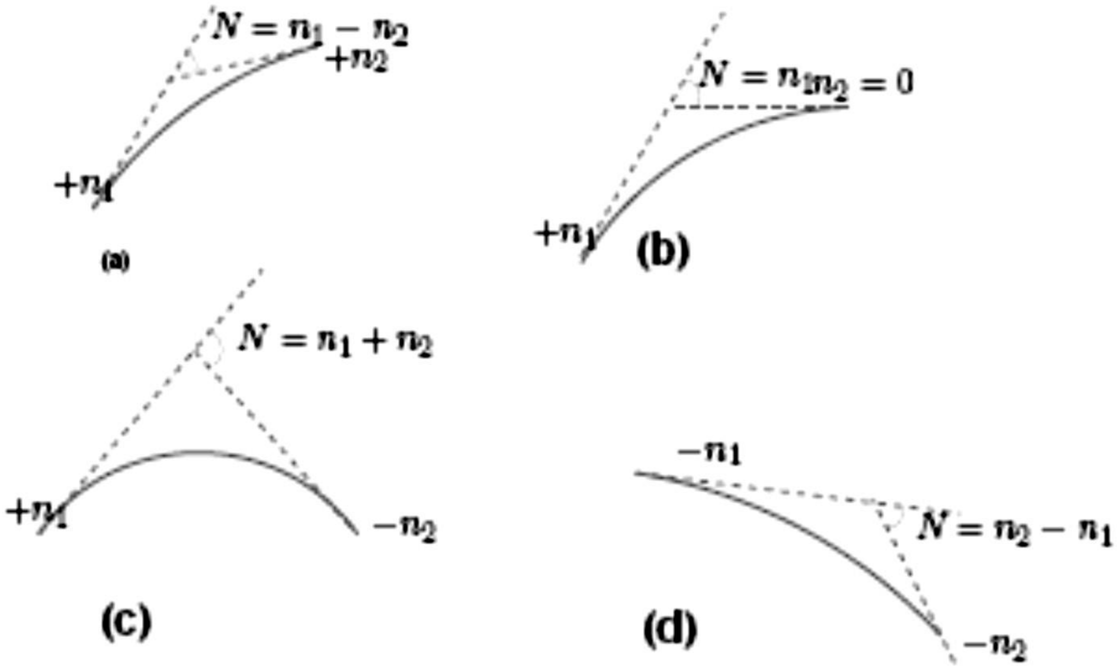


Fig. 2.11 Types of summit curves

2.5.2.1 Type of Summit Curve

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

Length of the summit curve The important design aspect of the summit curve is the determination of the length of the curve which is parabolic. As noted earlier, the length of the curve is guided by the sight distance consideration. Distance .Let L is the length

Case a: Length of summit curve greater than sight distance The situation when the sight distance is less than the length of the curve

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

Case b: Length of summit curve less than sight distance

When stopping sight distance is considered the height of driver's eye above the road surface (h1) is taken as 1.2 meters, and height of object above the pavement surface (h2) is taken as 0.15 meters. If overtaking sight distance is considered, then the value of driver's eye height (h1) and the height of the obstruction (h2) are taken equal as 1.2 meters.

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

2.5.3 Valley curve

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

1. When a descending gradient meets another descending gradient

2. When a descending gradient meets at gradient

3. When a descending gradient meets an ascending gradient

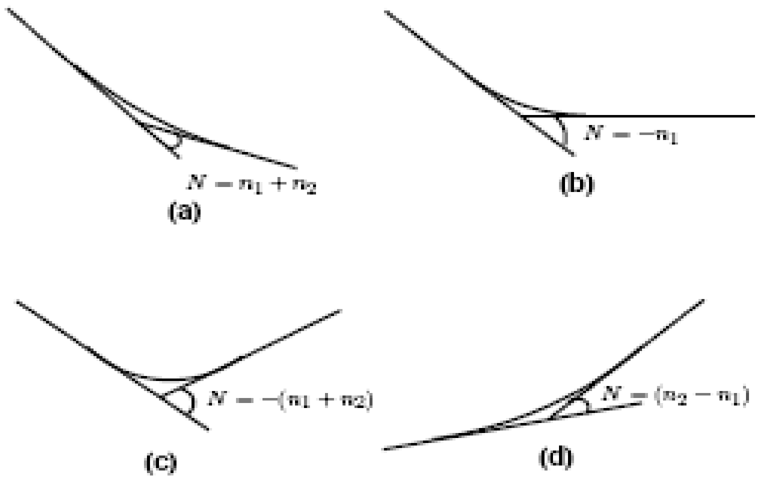


Fig. 2.12 Types of valley curve

2.5.3.1 Length of the valley curve

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

1. Comfort criteria; that is allowable rate of change of centrifugal acceleration is limited to a comfortable level of about $0.06 m/s^3$.

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

Safety criteria Length of the valley curve for headlight distance may be determined for two conditions: length of the valley curve greater than stopping sight distance and Length of the valley curve less than the stopping sight distance.

Case 1: Length of valley curve greater than stopping sight distance (L > S) The total length of valley curve L is greater than the stopping sight distance SSD. The sight distance available will be minimum when the vehicle is in the lowest point in the valley. This is because the beginning of the curve will have infinite radius and the bottom of the curve will have minimum radius which is a property of the transition curve.

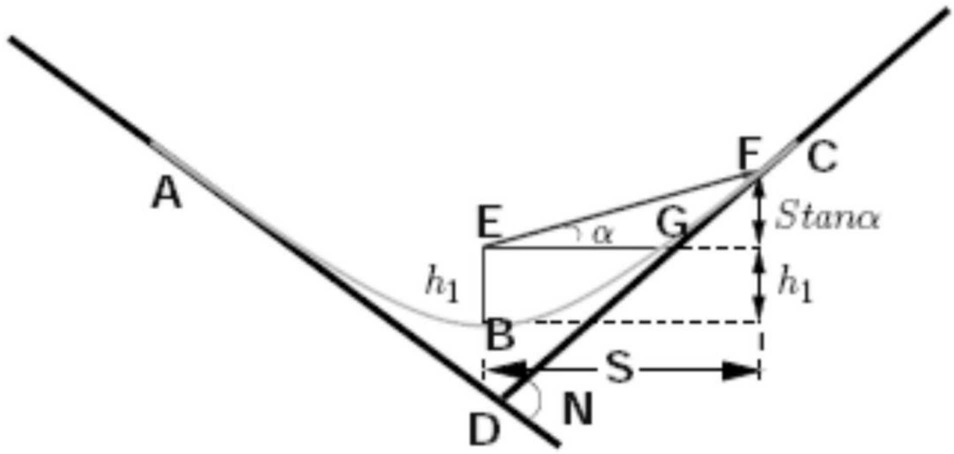


Fig. 2.13 Valley curve, case1, L > S

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 0.06 m/s^3.

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

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

Case 2 Length of valley curve less than stopping sight distance (L < S) The length of the curve L is less than SSD. In this case the minimum sight distance is from the beginning of the curve. The important points are the beginning of the curve and the bottom most part of the curve. If the vehicle is at the bottom of the curve, then its headlight beam will reach far beyond the endpoint of the curve whereas, if the vehicle is at the beginning of the curve, then the headlight beam will hit just outside the curve. Therefore, the length of the curve is derived by assuming the vehicle at the beginning of the curve. The case is shown in figure below.

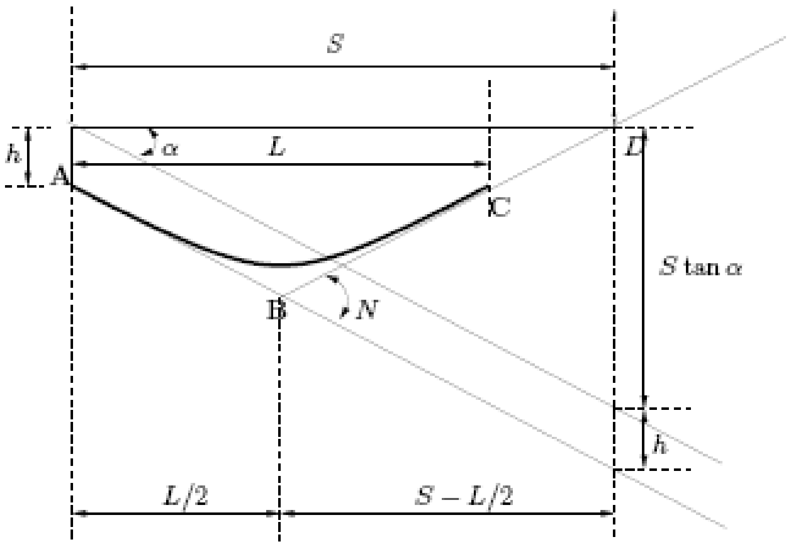


Fig. 2.14 Valley curve, case 2, S > L

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

UNIT-IV

Objective:

To learn standards, properties and laboratory tests of Highway Materials.

Syllabus: Highway materials: aggregate properties and tests: crushing, abrasion and impact test, bitumen properties and tests, penetration, ductility, viscosity, binder content and softening point. Marshall Method of mix design.

HIGHWAY MATERIALS

1.MATERIALS FOR FLEXIBLE PAVEMENT LAYERS:

The flexible pavement layers are constructed using the following basic materials:

i)selected granular soils or crushed aggregates or soil-aggregates mixes with adequate permeability in the drainage layer.

(ii) stone aggregates and fine aggregates in the granular base course

(iii) coarse aggregates, fine aggregates and bitumen binder in the bituminous pavement layers used in the base course or binder course and the surface course as per the design.

The rigid pavements are constructed using the following basic materials

(i)selected granular soil or crushed aggregates or soil aggregates mixes with adequate permeability in drainage layer.

(ii) coarse aggregates, fine aggregates and Portland cement for the lean cement concrete in sub-base course and

(ii) coarse aggregates, tine aggregates and Portland cement for preparation of pavement quality concrete in the cement concrete pavement slab, which serves as both base course and surface course.

Various other combinations of materials and stabilized mixes may also be used in different pavement layers, depending on their availability and design in place, some of the conventional pavement materials.

Different types of bituminous binders are used for the construction of flexible pavement layers. Portland cement is made use of for the construction of lean cement concrete sub-base course and paving quality concrete slab of the rigid pavements.

1.1.AGGREGATE PROPERTIES:

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

(a) Resistance to impact or toughness

(b)Resistance to abrasion or hardness.

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

(d)Resistance to crushing or crushing strength

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

(f) Resistance to weathering or durability

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

1.2.AGGREGATE TESTS:

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

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

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

(c) Polished stone value test or accelerated polishing test

(d) Aggregate crushing test (strength characteristics)

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

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

(g) Specific gravity test and water absorption test.

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

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

A.AGGREGATE CRUSHING VALUE:

The stone aggregates used for the construction of road pavements should possess satisfactory resistance to crushing under the roller during construction and under the application of heavy wheel loads on the pavement during its service life. The strength of coarse aggregate may be assessed by aggregate crushing test. The aggregate crushing value provides a relative measure of resistance to crushing under gradually applied compressive load. Aggregates possessing high resistance to crushing or low aggregate crushing value are preferred for use in high quality pavements. The apparatus for. the standard test consists of a steel cylinder 152 mm diameter with a base plate and a plunger, compression testing machines, cylindrical measure of diameter 115 mm and height 180 mm, tamping rod and sieves.

Dry aggregate passing 12.5 mm IS sieve and retained on 10 mm sieve is filled in the cylindrical measure in three equal layers, each layer being ramped 25 times by the tamper. The test sample is weighed (equal to W1 g) and placed in the test cylinder in three equal layers, tamping each layer 25 times. The plunger is placed on the top of specimen and a load of 40 tonnes is applied at a rate of 4 tonnes per minute by the compression machine. The crushed aggregate is removed and sieved on 2.36 mm IS sieve. The crushed material which passes this sieve is weighed equal to W2 g. The aggregate crushing value is the percentage of the crushed material passing 2.36 mm. The Sieve in terms of original weight of the specimen.100 W2

Aggregate crushing value = (100 W_2/W_1)* percent.

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

The aggregate crushing value for good quality aggregate to be used 1n base course shall not exceed 45 percent and the value for surface course shall be less than 30 percent. The IRC and BIS have specified that the aggregate crushing value of the Coarse aggregates to be used for cement concrete pavement surface should not exceed 30 percent. However aggregate crushing values have not been specified by the IRC or the Ministry of Road Transport and Highways for coarse aggregates to be used in flexible pavement/bituminous pavement construction methods.

FIG.1.2.1.AGGRAGATE CRUSHING TEST

B.LOS ANGELES ABRASION TEST:

The principle of Los Angeles abrasion test is to find the percentage wear due to the relative rubbing action between the aggregates and steel balls used as abrasive charge. During Los Angels abrasion test, both abrasion or rubbing action between the aggregates and the steel balls and also impact or pounding action of these balls on the aggregates takes place Therefore Los Angeles abrasion test is considered to be more reliable for evaluating suitability of coarse aggregates in pavements as both abrasion and impact occur during the test similar to the field conditions. This test has been standardised by the BIS, ASTM & AASHTO. Acceptable limits of Los Angeles abrasion values of coarse aggregates have been specified by the IRC and also the MORTH. The Los Angeles machine consists of a hollow cylinder closed at both ends, having inside diameter 700 mm and length 500 mm and mounted so as to rotate about its horizontal axis. A removable steel shelf projecting radially 88 mm into the cylinder and extending to the full length of it is mounted on the interior surface of the cylinder rigidly parallel to the axis. The abrasive charge consisting of cast iron spheres of approximate diameter

48 mm and each of weight 390 to 445 g is placed in the machine. The number of spheres to be used as abrasive charge and their total weight have been specified based on grading of the selected aggregate sample.

The BIS has specified seven sets of grading of coarse aggregates, namely grading A,B. C. D, E, F and G; for each grading different weights of aggregate specimen and abrasive charge have been specified. For grading - A, total 5.0 kg of coarse aggregates consisting of 1250 g each of size ranges, (i) 40 to 25 mm (ii) 25 to 20 mm (ii) 20 to 12.5 mm and (iv) 12.5 to 10 mm are placed in the machine along with abrasive charge consisting of 12 spheres of total weight (5000 g +/-25 g). For grading-B, total 5.0 kg of coarse aggregates consisting of 2500 g each of the coarse aggregates of size ranges, (i) 20 to 12.5 mm and (11) 12.5 to 10 mm are placed in the machine along with abrasive charge consisting of11 spheres of total weight (4584 g +/-25 g).The specified weight of aggregate specimen of desired grading is taken, (5 to 10 kg, depending on gradation) and placed in the machine along with the specified abrasive charge. The machine is rotated at a speed of 30 to 33 pm for the specified number of revolutions (500 to 1000 depending on the grading of the specimen). The abraded aggregate is then sieved on 1.7 mm IS sieve, and the weight of powdered aggregate passing this sieve is found. The result of the abrasion test expressed as the percentage wear or the percentage passing1.7 mm sieve expressed in terms of the original weight of the sample.

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

FIG.1.2.2.ABRASION TEST

C.AGGREGATE IMPACT TEST:

During the construction process of pavement layers, particularly compaction by heavy rollers and also due to movement of heavy wheel loads of traffic, the road aggregates are subjected to impact or pounding action and there is possibility of some stones breaking into smaller pieces. The stone aggregates should therefore be sufficiently tough to resist fracture under impact loads. This property could differ from the resistance to crushing of aggregates under gradually increasing compressive stress. The aggregate impact test is carried out to evaluate the resistance to impact of aggregates to fracture under repeated impacts; the test has been standardised by Bureau of Indian Standards (BIS). The aggregate impact testing machine consists of a metal base and a cylindrical steel cup of internal diameter 102 mm and depth 50 mm in which the aggregate specimen is placed. A cylindrical metal hammer of weight 13.5 to 14.0 kg having a a rom a height 380 mm is arranged to drop through vertical guides. Aggregate specimen passing 12.5 mm sieve and retained on 10 mm sieve is fill the cylindrical measure in three layers by tamping each layer by 25 blows by the tamping rod. The sample is weighed and transferred from the measure to the cup of the aggregate impact testing machine and compacted by tamping25 times. The hammer is raised to a height of 380 mm above the upper surface of the aggregate in the cup and is allowed to fall freely on the specimen. After subjecting the test specimen to15 blows, the crushed aggregate is sieved on 2.36 mm sieve. The aggregate impact value is expressed as the percentage of the fines passing 2.36 mm sieve formed in terms of the total weight of the sample. The above test is repeated using another specimen of the same aggregate sample. By taking the same weight as in the first test. The mean of the two test results is reported as the Aggregate Impact Value (AIV) of the specimen, to the nearest whole number.

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

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

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

FIG.1.2.3.AGGREGATE IMPACT TEST

2.BITUMEN PROPERTIES:

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

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

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

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

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

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

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

3.BITUMEN TESTS:

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

a. Penetration test
b. Viscosity test
c. Ductility test
d. Binder content
e. Softening point test
f. Specific gravity test
g. Flash and fire point tests
h. Loss on heating test
i. Solubility test

A.PENETRATION TEST:

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

The Penetration test is widely used for classifying the bitumen into different grades. The BIS has standardized the penetration test equipment and the test procedure. The penetration test determines the consistency of these materials for the purpose of grading them by measuring the depth to which a standard needle will penetrate. The vertically under specified conditions of standard load, duration and temperature. Thus the basic principle of the penetration test is the measurement of the penetration (in units of one tenth of a mm) of a standard needle in a bitumen sample maintained at 25C during five seconds, the total weight of the needle assembly being 100 g. Penetration test apparatus or the penetrometer consists of a penetration needle assembly which is attached to a calibrated dial. On release, the penetration needle penetrates into the bitumen specimen without appreciable friction. The bitumen is softened to a pouring consistency, stirred thoroughly and poured into containers to a depth at least 15 mm in excess of the expected penetration. The sample containers are then placed in a temperature controlled water bath at a temperature of 25°C for one hour. The sample with container is taken out, placed under the penetrometer and the needle is adjusted to make contact with the surface of the sample. The dial is set to zero or the initial reading is taken and the needle is released for 5 seconds. The final reading is taken on dial gauge. At least three penetration tests are made on this sample by testing at distances of at least 10 mm apart. After each test, the needle is disengaged and wiped with benzene and dried. The depth of penetration is reported in one-tenth mm units. The mean value of three measurements is reported as a penetration value. It may be noted that the penetration value is largely influenced by any inaccuracy as regards pouring temperature, size of needle, weight placed on the needle and the test temperature. Penetration test is the most commonly adopted to determine the grade of the bitumen in terms of its hardness because of its simplicity. Softer the bitumen, the greater will be the penetration value. 80/100 bitumen denotes that the penetration value. The penetration grades of bitumen binders of the binder ranges between 80 and 100. The penetration grades of bitumen binders are generally denoted as 80/100, 60/70 or 30/40 grade bitumen.

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

(i)penetration test is an empirical test and it has no relation with the fundamental properties of the binder

(ii)the test temperature of 25°C is not the general pavement service temperature

(iii) the service temperature of the pavement is much higher say, about 60°C for most part of the day in several regions

(iv) bitumen having the same penetration value may have different performance while in service depending on is temperature susceptibility; this is because bitumen having the same penetration value may have widely varying temperature-stiffness relationship.

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

FIG.3.1.PENETRATION TEST

B.DUCTILITY TEST:

In the flexible pavement constructions where bitumen binders are used, it is important that the binders form ductile thin films around the aggregates. The ductile film of binder improves the physical interlocking of the aggregate-bitumen mixes Under traffic loads, the bituminous pavement layer is subjected to repeated deformation and recoveries. The binder material which does not possess sufficient ductility would crack and permit the surface water to enter into the pavement resulting in rapid deterioration and failure. Ductility test is carried out on bitumen to test the adhesive property of bitumen and its ability to stretch. The bitumen may satisfy the penetration value, but may fail to satisfy the ductility requirements. The ductility value is expressed as the distance in centimetre (cm) to which the bitumen specimen of standard size can be stretched before the thread breaks. The standard briquette specimen has a minimum cross section 10 mmx 10 mm. The test is conducted at 27°C with a rate of pull of 50 mm per minute, until the stretched specimen breaks.

The ductility machine functions as a constant temperature water bath with a pulling device at a pre-calibrated rate. Two clips are thus pulled apart horizontally at a uniform speed of 50 mm per minute. The bitumen sample is heated and poured in the mould assembly placed on a plate. The samples along with the moulds are cooled in air and then in water bath maintained at 27°C. The excess bitumen material is trimmed and the surface is levelled using a hot knife. The mould assembly containing sample is replaced in water bath of the ductility testing machine for 85 to 95 minute. The sides of the mould are removed, the clips hooked on to the machine and the pointer is adjusted to zero. The distance up to the point of breaking of thread is reported as ductility value in cm. The ductility value gets seriously affected by factors such as pouring temperature, dimensions of briquette, level of briquette in the water bath, presence of air pockets in the specimen briquettes, test temperature and rate of pulling. The ductility values of bitumen generally vary from 5 to over 100 for different bitumen grades. A minimum ductility value of 50 to 75 cm is generally specified for bitumen used in pavement construction.

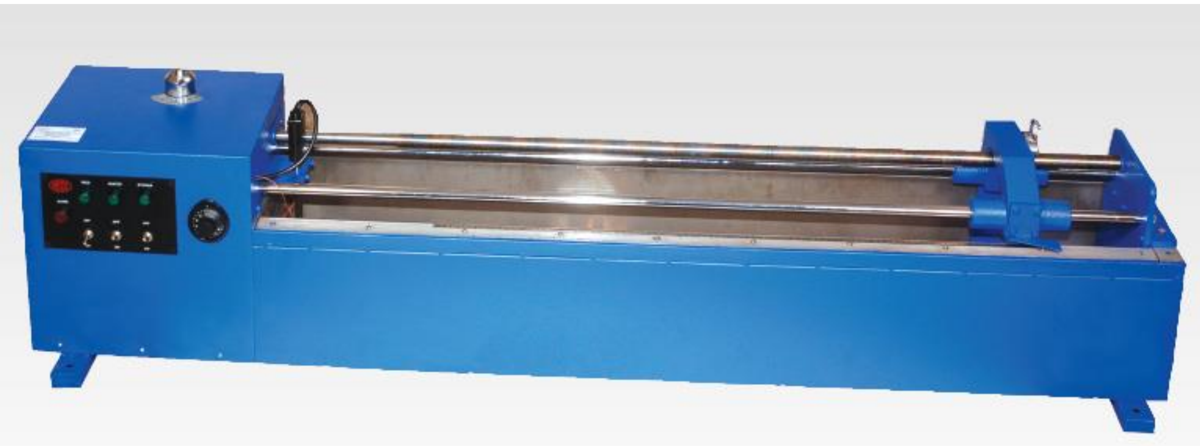


FIG.3.2.DUCTILITY TEST

FIG.3.3 CONCEPT OF DUCTILITY TEST

C.VISCOSITY TEST:

Viscosity is indirectly measured by determining the time taken by 50 ml of the binder in fluid state to flow through a specified orifice from a cup, under standard test conditions and specified temperature. This method is suitable for measuring viscosity of bitumen emulsion, cut-back bitumen and tar. Determination of viscosity using orifice viscometer Viscosity of liquid bituminous binders like bitumen emulsion and tar determined by indirect method using orifice type viscometers. A specified quantity of the binder (50 ml) is allowed to flow through specified orifice size of the test-cup at a given temperature and the time taken in seconds is recorded as the viscosity value. As per the specifications of Bureau of Indian Standards, the viscosity values of bitumen emulsions are determined using Saybolt Furol' orifice viscometer at test temperatures of 25 C and 50 C. The viscosity values of tar are determined using orifice viscometer called Tar Viscometer using either 10 mm or 4 mm size orifice.

FIG.3.4. VISCOSITY TEST MACHINE

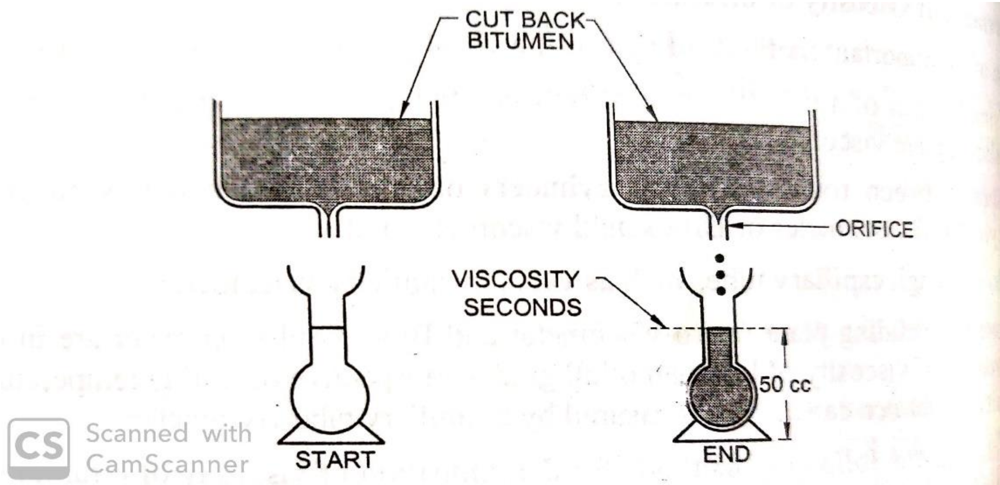


FIG.3.4.1 CONCEPT OF VISCOSITY TEST

D.BINDER CONTENT:

To determine the binder content in the asphalt mix by cold solvent extraction

Apparatus which are used in this experiment are:

Centrifuge, Balance of capacity 500 gram and sensitivity 0.01grams, Thermostatically controlled oven with capacity up to 2500 C,Beaker for collecting extracted material.

TEST PROCEDURE :

Take exactly 500 grams of representative sample and place in the bowl of extraction apparatus (W1). Add benzene to the sample until it is completely submerged. Dry and weigh the filter paper and place it over the bowl of the extraction apparatus containing the sample (F1) . Clamp the cover of the bowl tightly. Place a beaker under the drainpipe to collect the extract Sufficient time (not more than an hour) is allowed for the solvent to disintegrate the sample before running the centrifuge. Bitumen Extractor. Run the centrifuge slowly and then gradually increase the speed to a maximum of 3600 rpm Maintain the same speed till the solvent ceases to flow from the drainpipe. Run the centrifuge until the bitumen and benzene are drained out completely. Stop the machine, remove the cover and add 200ml of benzene to the material in the extraction bowl and the extraction is done in the same process as described above. Repeat the same process not less than three times till the extraction is clear and not darker than a light straw colour. Collect the material from the bowl of the extraction machine along with the filter paper and dry it to constant weight in the oven at a temperature of 1050 C to 1100 C and cool to room temperature. Weigh the material (W2) and the filter paper (F2) separately to an accuracy of 0.01grams.

CALCULATIONS

- Percentage of binder in the total mix = [W1 – (W2 + W3)] x 100 W1

W1 = Weight of sample taken, W2 = Weight of sample after extraction, W3 = Increased weight of filter paper (F2 – F1)

FIG.3.5 BINDER CONTENT MACHINE

E.SOFTENING POINT:

The softening point is the temperature at which the substance attains a particular degree of softening under specified condition of test. The softening point of bitumen is usually determined by Ring and Ball test. The concept of softening point test and the test set-up is shown in following Fig. Generally higher softening point indicates lower temperature susceptibility and is, preferred in warm climates. A brass ring containing test sample of bitumen is suspended in a beaker with liquid bath; water is used as the bath if the softening point is less than 80”C and glycerine is used for temperatures exceeding 80°C. A steel ball is placed upon the bitumen sample and the liquid medium is then heated at a rate of 5°C per minute. The temperature at which the softened bitumen touches the metal plate placed at a specified distance below the ring is recorded as the softening point of the bitumen. Harder grades of bitumen possess higher softening point than soft grade bitumen. The softening point of various bitumen grades used in paving jobs vary between35° to 70°C.

FIG.3.6 SOFTENING POINT

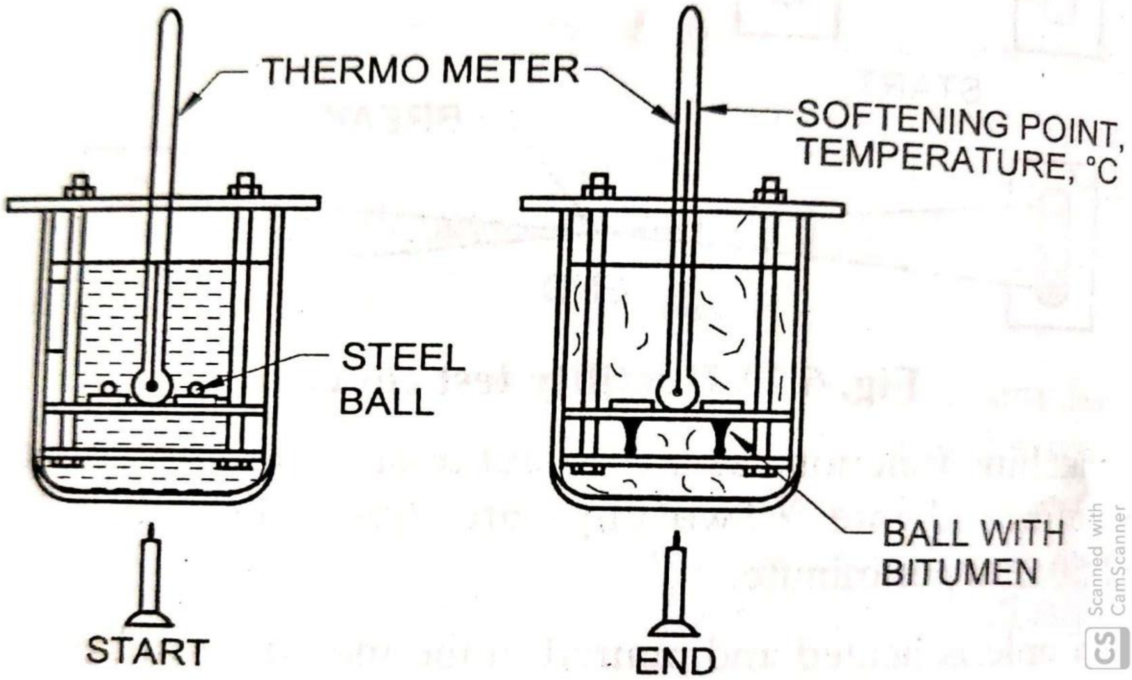


FIG.3.6.1 CONCEPT OF SOFTENING POINT

F.MARSHALL METHOD OF MIX DESIGN:

Marshall Stability test is conducted on compacted cylindrical specimens of bituminous mix of diameter 101.6 mm and thickness 63.5 mm. The load is applied perpendicular to the axis of the cylindrical specimen through a testing head consisting of a pair of cylindrical segments, at a constant rate of deformation of 51 mm per minute at the standard test temperature of 60°C.

The Marshall Stability' of the bituminous mix specimen is defined as a maximum load carried in kg at the standard test temperature of 60 °C when load is applied under specified test conditions. The Flow Value' is the total deformation that the Marshall The Marshall test specimen under-goes at the maximum load, expressed in mm units. The Marshall stability value of a compacted specimen of bituminous mix indicates its resistance to deformation under applied incremental load and the flow value indicates the extent of deformation it undergoes due to loading or its 'flexibility'.

Suggested procedure for mix design:

The steps for the design of bituminous mix by Marshall method are given below

(a) Specified/desired grading of the mix is selected from the recommended gradation for the particular type of pavement layer

(b) Representative samples of aggregates of different sizes proposed to be used in project are collected from the site of the hot mix plant or crusher (These aggregates should be from approved quarries fulfilling the specified physical requirements)

(c) Sieve analysis is carried out on the samples of the aggregates collected and the proportion in which they should be mixed to obtain desired gradation is determined (by any one of the methods, such as graphical method or trial method)

(d) The specific gravity of the coarse and fine aggregates and the bituminous binder used are determined.

(e) Adequate quantity of the aggregates and mineral filler are collected and mixed in the desired proportion as (c) above

(f) Five to six bitumen contents to be used in the trial mixes are selected so as to cover at least two values each below and above the probable/estimated value of optimum binder content (which depends on the gradation of the aggregates selected and the fines content)

(g) Marshall stability test specimens are prepared by compacting in the mould with specified number of blows, using the different percentages of bitumen content (with at least three test specimens at each bitumen content).

(h) The weight and mean dimensions or volume are determined for each specimen

(i) The values of percentage air voids (V), voids in mineral aggregates (VMA) and the voids filled with bitumen (VFB) are calculated for each test specimen and the mean of these for specimens prepared using different binder contents are tabulated

(j) Marshall stability test is conducted on each specimen and the mean of Marshall stability value (after applying the correction factor if any) and flow value for specimens prepared using different binder contents are tabulated.

(k) Graphs are plotted with bitumen content on the X axis and (i) density (ii)Marshall stability (iii) flow value (iv) air voids (v) VFB and (vi) VMA on the Y- axis

(l) Individual values of optimum bitumen contents are obtained considering (maximum density (ii) maximum stability (iii) mid range of recommended flow value (iv) mid range of recommended voids content

(m) Considering the different values of optimum bitumen contents determined as above, a suitable design bitumen content is selected within the range of optimum values mentioned. Corresponding to the selected value of design bitumen content, the values of Marshall stability, flow, and air voids in the mix are noted from the graphs and are checked to find if they fulfil the specified mix design criteria

(n) If required, another value of design binder content may be tried using the same set of test values and graphs. If all the specified design criteria could not be fulfilled with the selected gradation/proportion of aggregates, the mix design tests may be repeated after altering the gradation/proportion of different aggregates.

(o) The proportion of mixing different aggregates, filler and bitumen are specified by weight or by volume for implementation during construction as the job mix formula.

FIG.3.7. MARSHALL STABILITY TESTING MACHINE

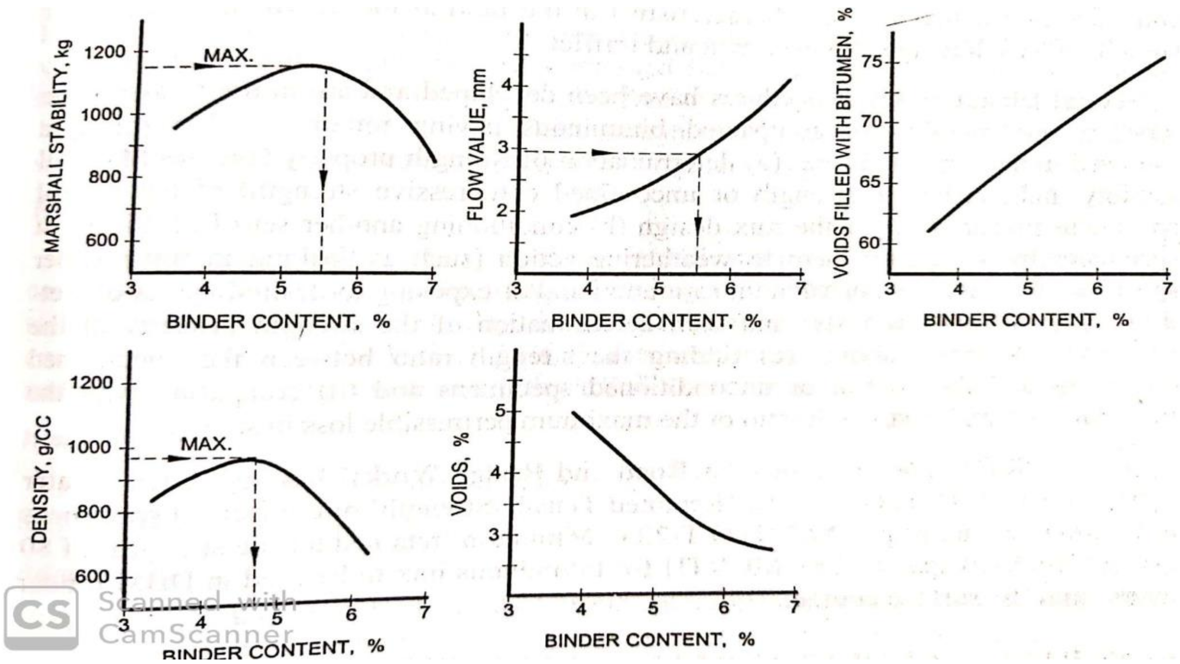


FIG.3.7.1.BITUMINOUS MIX DESIGN NY MARSHALL METHOD

LEARNING MATERIAL UNIT 5

UNIT - V: Design of Pavements and Pavement Failures

Design of flexible pavement by CBR method as per IRC 37-2012, stresses in rigid pavement by Westergaards and IRC methods. Failures in flexible pavements, Failures in rigid pavements

FLEXIBLE PAVEMENT DESIGN BY IRC GUIDELINES:

Failure Criteria

A and B are the critical locations for tensile strains (ϵ_t). Maximum value of the strain is adopted for design. C is the critical location for the vertical subgrade strain (ϵ_z) since the maximum value of the (ϵ_z) occurs mostly at C.

Fatigue Criteria:

Bituminous surfacings of pavements display flexural fatigue cracking if the tensile strain at the bottom of the bituminous layer is beyond certain limit. The relation between the fatigue life of the pavement and the tensile strain in the bottom of the bituminous layer was obtained as

$$N_f = 2.21 \times 10^{-4} \times \left(\frac{1}{\epsilon_t}\right)^{3.89} \times \left(\frac{1}{E}\right)^{0.854}$$

in which, N_f is the allowable number of load repetitions to control fatigue cracking and E is the Elastic modulus of bituminous layer. The use of equation 28.1 would result in fatigue cracking of 20% of the total area.

Rutting Criteria

The allowable number of load repetitions to control permanent deformation can be expressed as

$$N_r = 4.1656 \times 10^{-8} \times \left(\frac{1}{\epsilon_z}\right)^{4.5337}$$

N_r is the number of cumulative standard axles to produce rutting of 20 mm.

Design procedure

Based on the performance of existing designs and using analytical approach, simple design charts and a catalogue of pavement designs are added in the code. The pavement designs are given for subgrade CBR values ranging from 2% to 10% and design traffic ranging from 1 msa to 150 msa for an average annual pavement temperature of 35 C. The later thicknesses obtained from the analysis have been slightly modified to adapt the designs to stage construction. Using the following simple input parameters, appropriate designs could be chosen for the given traffic and soil strength:

- Design traffic in terms of cumulative number of standard axles; and
- CBR value of subgrade.

Design traffic

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

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

Initial traffic

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

Traffic growth rate

Traffic growth rates can be estimated (i) by studying the past trends of traffic growth, and (ii) by establishing econometric models. If adequate data is not available, it is recommended that an average annual growth rate of 7.5 percent may be adopted.

Design life

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

Vehicle Damage Factor

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

Vehicle distribution

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

- **Single lane roads:** Traffic tends to be more channelized on single roads than two lane roads and to allow for this concentration of wheel load repetitions, the design should be based on total number of commercial vehicles in both directions.
- **Two-lane single carriageway roads:** The design should be based on 75 % of the commercial vehicles in both directions.
- **Four-lane single carriageway roads:** The design should be based on 40 % of the total number of commercial vehicles in both directions.
- **Dual carriageway roads:** For the design of dual two-lane carriageway roads should be based on 75 % of the number of commercial vehicles in each direction. For dual three-lane carriageway and dual four-lane carriageway the distribution factor will be 60 % and 45 % respectively.

Computation of design traffic

The design traffic, in terms of the cumulative number of standard axles to be carried during the design period of the road, should be estimated using equation 4.5.

$$N_{Des} = \frac{365 \times [(1+r)^n - 1]}{r} \times A \times D \times F$$

Where,

N_{Des} = cumulative number of standard axles to be catered for during the design period of 'n' years

A = initial traffic (commercial vehicles per day) in the year of completion of construction (directional traffic volume to be considered for divided carriageways where as for other categories of the carriageway, two-way traffic volume may be considered for applying the lateral distribution factors)

D = lateral distribution factor (as explained in para 4.5)

F = vehicle damage factor (VDF)

n = design period, in years

r = annual growth rate of commercial vehicles in decimal (e.g., for 6 per cent annual growth rate, r = 0.06). Variation of the rate of growth over different periods of the design period, if available, may be considered for estimating the design traffic

The traffic in the year of completion of construction may be estimated using equation 4.6.

$$A = P(1 + r)^x \tag{4.6}$$

Where,

P = number of commercial vehicles per day as per last count.

x = number of years between the last count and the year of completion of construction.

4.6.2 For single carriageway (undivided) roads, the pavement may be designed for design traffic estimated based on the larger of the two VDF values obtained for the two directions. For divided carriageways, different pavement designs can be adopted for the two directions of traffic depending on the directional distribution of traffic and the corresponding directional VDF values in the two directions.

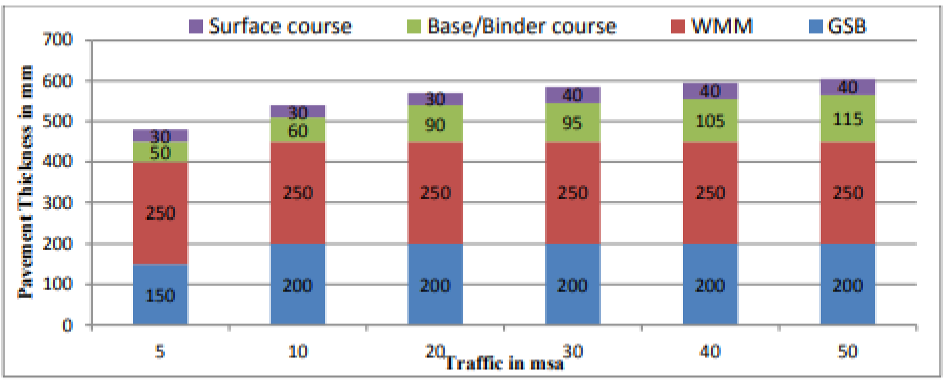


Figure 12.4 Catalogue for pavement with bituminous surface course with granular base and sub-base - Effective CBR 8% (Plate-4)

Stresses in rigid pavement by Westergaards and IRC methods:

As the name implies, rigid pavements are rigid i.e, they do not ex much under loading like exible pavements. They are constructed using cement concrete. In this case, the load carrying capacity is mainly due to the rigidity ad high modulus of elasticity of the slab (slab action). H. M. Westergaard is considered the pioneer in providing the rational treatment of the rigid pavement analysis.

Modulus of sub-grade reaction:

Westergaard considered the rigid pavement slab as a thin elastic plate resting on soil sub-grade, which is assumed as a dense liquid. The upward reaction is assumed to be proportional to the deflection. Base on this assumption, Westergaard defined a modulus of sub-grade reaction K in kg/cm3 given by K = P/Δ where Δ is the displacement level taken as 0.125 cm and p is the pressure sustained by the rigid plate of 75 cm diameter at a deflection of 0.125 cm.

Relative stiffness of slab to sub-grade

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

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

where E is the modulus of elasticity of cement concrete in kg/cm^2 (3.0×10^5), μ is the Poisson's ratio of concrete (0.15), h is the slab thickness in cm and K is the modulus of sub-grade reaction.

Critical load positions:

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

Equivalent radius of resisting section:

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

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

where a is the radius of the wheel load distribution in cm and h is the slab thickness in cm

Wheel load stresses - Westergaard's stress equation:

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

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

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

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

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

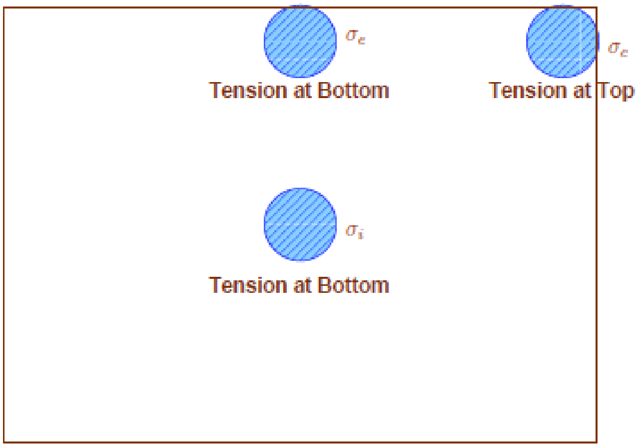


Figure : Critical stress locations

Temperature stresses:

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

(i) daily variation resulting in a temperature gradient across the thickness of the slab and

(ii) seasonal variation resulting in overall change in the slab temperature. The former results in **warping stresses** and the later in **frictional stresses.**

Warping stress

The warping stress at the interior, edge and corner regions, denoted as σ_{tc},σ_{te},σ_{ti} in kg/cm^2 respectively and given by the equation

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

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

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

where **E** is the modulus of elasticity of concrete in kg/cm^2 ($3x10^5$), **ϵ** is the thermal coefficient of concrete per °C ($1x10^{-7}$) **t** is the temperature difference between the top and bottom of the slab, **Cx** and **Cy** are the coefficient based on Lx/l in the desired direction and Ly/l right angle to the desired direction, **μ** is the Poisson's ratio (0.15), **a** is the radius of the contact area and **l** is the radius of the relative stiffness.

Frictional stresses:

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

$$\sigma_f = \frac{WLf}{2 \times 10^4}$$

where **W** is the unit weight of concrete in kg/cm^2 (2400), **f** is the coefficient of sub grade friction (1.5) and **L** is the length of the slab in meters.

Combination of stresses

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

- Summer, mid-day: The critical stress is for edge region given by

$$\sigma_{critical} = \sigma_e + \sigma_{t_e} - \sigma_f$$

- Winter, mid-day: The critical combination of stress is for the edge region given by

$$\sigma_{critical} = \sigma_e + \sigma_{t_e} + \sigma_f$$

- Mid-nights: The critical combination of stress is for the corner region given by

$$\sigma_{critical} = \sigma_c + \sigma_{t_c}$$

Failures in flexible pavements:

TYPES OF FLEXIBLE PAVEMENT FAILURES

1. Fatigue (alligator) cracking

2. Bleeding

3. Block cracking

4. Corrugation and shoving

5. Depression

6. Joint reflection cracking

7. Lane/shoulder drop-off

8. Longitudinal cracking

9. Patching

10. Polished aggregate

11. Pothole

12. Ravelling

13. Rutting

14. Slippage Cracking

15. Stripping

16. Transverse (thermal)cracking

17. Water bleeding and pumping

1.FATIGUE (ALLIGATOR) CRACKING Series of interconnected cracks caused by fatigue failure of the Hot Mix laid surface (or stabilized base) under repeated traffic loading. In thin pavements, cracking initiates at the bottom of the Hot Mix layer where the tensile stress is the highest then propagates to the surface as one or more longitudinal cracks. This is commonly referred to as "bottom-up" or "classical" fatigue cracking. In thick pavements, the cracks most likely initiate from the top in areas of high localized tensile stresses resulting from tire-pavement interaction and bitumen binder aging . After repeated loading, the longitudinal cracks connect forming many-sided sharp-angled pieces that develop into a pattern resembling the back of an alligator or crocodile.

Possible Causes:

Inadequate structural support, which can be caused by a myriad of things. A few of the more common ones are listed here: • Decrease in pavement load supporting characteristics

• Loss of base, sub base or Subgrade support e.g., poor drainage

• Stripping on the bottom of the Hot Mix layer the stripped portion contributes little to pavement strength so the effective Hot Mix thickness decreases.

• Increase in loading e.g., more or heavier loads than anticipated in design.

• Inadequate structural design Poor construction e.g., inadequate compaction.

Repairs: A fatigue cracked pavement should be investigated to determine the root cause of failure. Any investigation should involve digging a pit or coring the pavement to determine the pavement's structural makeup as well as determining whether or not subsurface moisture is a contributing factor. Once the characteristic alligator pattern is apparent, repair by crack sealing is generally ineffective. Fatigue crack repair generally falls into one of two categories: Small, localized fatigue cracking indicative of a loss of Subgrade support. Remove the cracked pavement area then dig out and replace the area of poor Subgrade and improve the drainage of that area if necessary. Patch over the repaired Subgrade. Large fatigue cracked areas indicative of general structural failure. Place an Hot Mix overlay over the entire pavement surface. This overlay must be strong enough structurally to carry the anticipated loading because the underlying fatigue cracked pavement most likely contributes little or no strength.

2.Bleeding A film of bitumen binder on the pavement surface. It usually creates a shiny, glass-like reflecting surface that can become quite sticky.

Problem: Loss of skid resistance when wet

Possible Causes: Bleeding occurs when bitumen binder fills the aggregate voids during hot weather and then expands onto the pavement surface. Since bleeding is not reversible during cold weather, bitumen binder will accumulate on the pavement surface over time.

This can be caused by one or a combination of the following:

• Excessive bitumen binder in the Hot Mix (either due to mix design or manufacturing)

• Excessive application of bitumen binder during application

• Low Hot Mix air void content (e.g., not enough room for the bitumen to expand into during hot weather)

Repair: The following repair measures may eliminate or reduce the bitumen binder film on the pavement's surface but may not correct the underlying problem that caused the bleeding:

• Minor bleeding can often be corrected by applying coarse sand to blot up the excess bitumen binder.

• Major bleeding can be corrected by cutting off excess bitumen with a motor grader or removing it with a heater planer.

• If the resulting surface is excessively rough, resurfacing may be necessary.

3.Block Cracking: Interconnected cracks that divide the pavement up into rectangular pieces. Blocks range in size from approximately 0.1 m^2 to 9 m^2 . Larger blocks are generally classified as longitudinal and transverse cracking. Block cracking normally occurs over a large portion of pavement area but sometimes will occur only in non-traffic areas. Problem: Allows moisture infiltration, roughness

Possible Causes: Hot Mix layer shrinkage and daily temperature cycling. Typically caused by an inability of bitumen binder to expand and contract with temperature cycles because of:

- Bitumen binder aging

- Poor choice of bitumen binder in the mix design

Repairs on Block Cracking: Strategies depend upon the severity and extent of the block cracking: • *Low severity cracks (< 1/2 inch wide)*. Crack seal to prevent

(1) entry of moisture into the Subgrade through the cracks.

(2) further raveling of the crack edges. Hot Mix can provide years of satisfactory service after developing small cracks if they are kept sealed.

• *High severity cracks (> 1/2 inch wide and cracks with raveled edges)*. Remove and replace the cracked pavement layer with an overlay.

4.Corrugation and Shoving A form of plastic movement typified by ripples (corrugation) or an abrupt wave (shoving) across the pavement surface. The distortion is perpendicular to the traffic direction. Usually occurs at points where traffic starts and stops (corrugation) or areas where Hot Mix abuts a rigid object (shoving).

Problem: Roughness Possible

Causes: Usually caused by traffic action (starting and stopping) combined with:

• An unstable (i.e. low stiffness) Hot Mix layer (caused by mix contamination, poor mix design, poor Hot Mix manufacturing, or lack of aeration of liquid bitumen emulsions)

• Excessive moisture in the Subgrade.

Repair: A heavily corrugated or shoved pavement should be investigated to determine the root cause of failure. Repair strategies generally fall into one of two categories:

• *Small, localized areas of corrugation or shoving*. Remove the distorted pavement and patch.

• *Large corrugated or shoved areas indicative of general Hot Mix failure*. Remove the damaged pavement and overlay.

5.Depression Localized pavement surface areas with slightly lower elevations than the surrounding pavement. Depressions are very noticeable after a rain when they fill with water.

Problem: Roughness, depressions filled with substantial water can cause vehicle hydroplaning.

Possible Causes: Frost heave or Subgrade settlement resulting from inadequate compaction during construction.

Repair: By definition, depressions are small localized areas. A pavement depression should be investigated to determine the root cause of failure (i.e., Subgrade settlement).

Depressions should be repaired by removing the affected pavement then digging out and replacing the area of poor Subgrade. Patch over the repaired Subgrade.

6.Joint Reflection Cracking Cracks in a flexible overlay of a rigid pavement. The cracks occur directly over the underlying rigid pavement joints. Joint reflection cracking does not include reflection cracks that occur away from an underlying joint or from any other type of base (e.g., cement or lime stabilized).

Problem: Allows moisture infiltration, roughness

Possible Causes: Movement of the PCC slab beneath the Hot Mix surface because of thermal and moisture changes. Generally not load initiated, however loading can hasten deterioration.

Repair: Strategies depend upon the severity and extent of the cracking:

• ***Low severity cracks*** *(< 1/2 inch wide and infrequent cracks)*. Crack seal to prevent

(1) entry of moisture into the Subgrade through the cracks and

(2) further raveling of the crack edges. In general, rigid pavement joints will eventually reflect through an Hot Mix overlay without proper surface preparation.

• ***High severity cracks*** *(> 1/2 inch wide and numerous cracks)*. Remove and replace the cracked pavement layer with an overlay.

7.Longitudinal Cracking Cracks parallel to the pavement's centreline or lay down direction. Usually a type of fatigue cracking.

Problem: Allows moisture infiltration, roughness, indicates possible onset of alligator cracking and structural failure.

Possible Causes:

• Poor joint construction or location. Joints are generally the least dense areas of a pavement. Therefore, they should be constructed outside of the wheel path so that they are only infrequently loaded.

Repair: Strategies depend upon the severity and extent of the cracking:

• ***Low severity cracks*** *(< 1/2 inch wide and infrequent cracks)*. Crack seal to prevent entry of moisture into the Subgrade through the cracks and further raveling of the crack edges. Hot Mix can provide years of satisfactory service after developing small cracks if they are kept sealed.

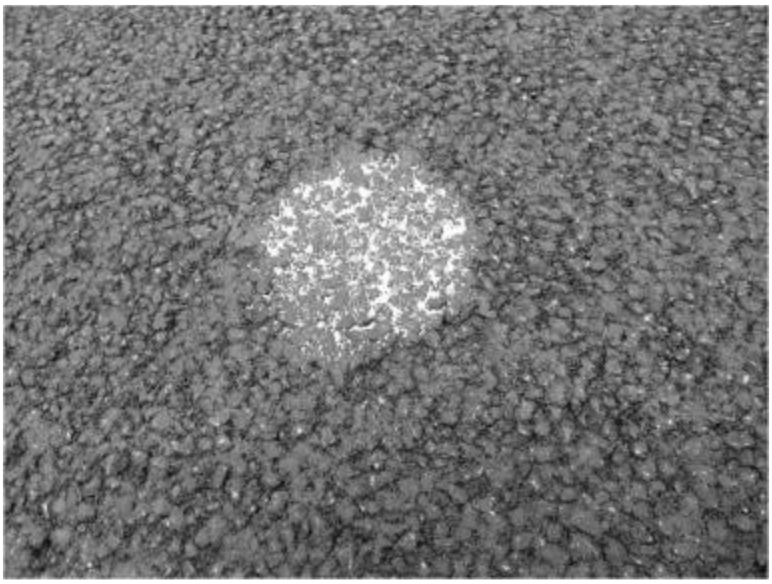
• ***High severity cracks*** *(> 1/2 inch wide and numerous cracks)*. Remove and replace the cracked pavement layer with an overlay.

8.Patching An area of pavement that has been replaced with new material to repair the existing pavement. A patch is considered a defect no matter how well it performs.

Problem: Roughness Possible Causes: Previous localized pavement deterioration that has been removed and patched Utility cuts.

Repair: Patches are themselves a repair action. The only way they can be removed from a pavement's surface is by overlay.

9.Polished Aggregate Areas of Hot Mix pavement where the portion of aggregate extending above the bitumen binder is either very small or there are no rough or angular aggregate particles.



Problem: Decreased skid resistance Possible Causes: Repeated traffic applications. Generally, as a pavement ages the protruding rough, angular particles become polished. This can occur quicker if the aggregate is susceptible to abrasion or subject to excessive studded tire wear.

Repair: Apply a skid-resistant slurry seal or overlay.

10. Potholes

Small, bowl-shaped depressions in the pavement surface that penetrate all the way through the Hot Mix layer down to the base course. They generally have sharp edges and vertical sides near the top of the hole. Potholes are most likely to occur on roads with thin Hot Mix surfaces (25 to 50 mm (1 to 2 inches)) and seldom occur on roads with 100 mm (4 inch) or deeper Hot Mix surfaces.

Problem: Roughness (serious vehicular damage can result from driving across potholes at higher speeds), moisture infiltration. Possible Causes: Generally, potholes are the end result of alligator cracking. As alligator cracking becomes severe, the interconnected cracks create small chunks of pavement, which can be dislodged as vehicles drive over them. The remaining hole after the pavement chunk is dislodged is called a pothole.

Repair: In accordance with patching techniques.

11.Raveling The progressive disintegration of an Hot Mix layer from the surface downward as a result of the dislodgement of aggregate particles.

Problem: Loose debris on the pavement, roughness, water collecting in the raveled locations resulting in vehicle hydroplaning, loss of skid resistance.

Possible Causes: Loss of bond between aggregate particles and the bitumen binder as a results as a dust coating on the aggregate particles that forces the bitumen binder to bond with the dust rather than the aggregate. Aggregate Segregation. If fine particles are missing from the aggregate matrix, then the bitumen binder is only able to bind the remaining coarse particles at their relatively few contact points. Inadequate compaction during construction. High density is required to develop sufficient cohesion within the Hot Mix. The third figure above shows a road suffering from raveling due to inadequate compaction caused by cold weather paving.

Repair: A raveled pavement should be investigated to determine the root cause of failure. Repair strategies generally fall into one of two categories:

• *Small localized areas of raveling*. Remove the raveled pavement and patch.

• *Large raveled areas indicative of general Hot Mix laid failure*. Remove the damaged pavement and overlay.

12.Rutting Surface depression in the wheel path. Pavement uplift (shearing) may occur along the sides of the rut. Ruts are particularly evident after a rain when they are filled with water. There are two basic types of rutting: mix rutting and Subgrade rutting. Mix rutting occurs when the Subgrade does not rut yet the pavement surface exhibits wheel path depressions as a result of compaction/mix design problems. Subgrade rutting occurs when the Subgrade exhibits wheel path depressions due to loading. In this case, the pavement settles into the Subgrade ruts causing surface depressions in the wheel path.

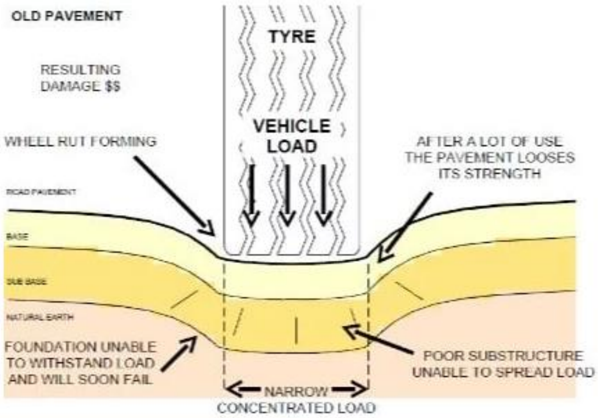
Problem: Ruts filled with water can cause vehicle hydroplaning, can be hazardous because ruts tend to pull a vehicle towards the rut path as it is steered across the rut.

Possible Causes: Permanent deformation in any of a pavement's layers or Subgrade usually caused by consolidation or lateral movement of the materials due to traffic loading. Specific causes of rutting can be:

• Insufficient compaction of Hot Mix layers during construction. If it is not compacted enough initially, Hot Mix pavement may continue to densify under traffic loads in the wheel path.

• Improper mix design or manufacture (e.g., excessively high bitumen content, excessive mineral filler, insufficient amount of angular aggregate particles)

• Ruts caused by studded tyre wear present the same problem as the ruts described here, but they are actually a result of mechanical dislodging due to wear and not pavement deformation.



the Rutting Formation under Vehicular Load

Real-Time Formation of Ruts

Repair:

• A heavily rutted pavement should be investigated to determine the root cause of failure (e.g. insufficient compaction, Subgrade rutting, poor mix design or studded tyre wear).

• Slight ruts (< 1/3 inch deep) can generally be left untreated. Pavement with deeper ruts should be levelled and overlaid.

13.Slippage Cracking Crescent or half-moon shaped cracks generally having two ends pointed into the direction of traffic.

Problem: Allows moisture infiltration, roughness.

Possible Causes: Braking or turning wheels cause the pavement surface to slide and deform. The resulting sliding and deformation is caused by a low-strength surface mix or poor bonding between the surface Hot Mix layer and the next underlying layer in the pavement structure.

Repair: Removal and replacement of affected area.

14.Stripping: The loss of bond between aggregates and bitumen binder that typically begins at the bottom of the Hot Mix layer and progresses upward. When stripping begins at the surface and progresses downward it is usually called raveling.

Problem: Decreased structural support, rutting, shoving/corrugations, raveling, or cracking (alligator and longitudinal)

Possible Causes: Bottom-up stripping is very difficult to recognize because it manifests itself on the pavement surface as other forms of distress including rutting, shoving/corrugations, raveling, or cracking. Typically, a core must be taken to positively identify stripping as a pavement distress.

• Poor aggregate surface chemistry

• Water in the Hot Mix laid causing moisture damage

• Overlays over an existing open-graded surface course., these overlays will tend to strip.

Repair: A stripped pavement should be investigated to determine the root cause of failure (i.e., how did the moisture get in). Generally, the stripped pavement needs to be removed and replaced after correction of any subsurface drainage issues.

15.Transverse (Thermal) Cracking Cracks perpendicular to the pavement's centreline or lay down direction. Usually a type of thermal cracking.

Problem: Allows moisture infiltration, roughness.

Possible Causes: Several including:

• Shrinkage of the Hot Mix laid surface due to low temperatures or bitumen binder hardening

• Reflective crack caused by cracks beneath the surface Hot Mix layer top-down cracking.

Repair: Strategies depend upon the severity and extent of the cracking:

• *Low severity cracks (< 1/2 inch wide and infrequent cracks)*. Crack seal to prevent

(1) entry of moisture into the Subgrade through the cracks and

(2) further raveling of the crack edges. Hot Mix laid can provide years of satisfactory service after developing small cracks if they are kept sealed.

• *High severity cracks (> 1/2 inch wide and numerous cracks)*. Remove and replace the cracked pavement layer with an overlay.

16.Water Bleeding and Pumping Water bleeding occurs when water seeps out of joints or cracks or through an excessively porous Hot Mix layer. Pumping occurs when water and fine material is ejected from underlying layers through cracks in the Hot Mix layer under moving loads.

Problem: Decreased skid resistance, an indication of high pavement porosity (water bleeding), decreased structural support (pumping)

Possible Causes: Several including:

• Porous pavement as a result of inadequate compaction during construction or poor mix design

• High water table

• Poor drainage

Repair: Water bleeding or pumping should be investigated to determine the root cause. If the problem is a high water table or poor drainage, Subgrade drainage should be improved. If the problem is a porous mix (in the case of water bleeding) a fog seal or slurry seal may be applied to limit water infiltration.

RIGID PAVEMENT FAILURES:

1.Spalling Cracking, breaking or chipping of joint/crack edges. Usually occurs within about 0.6 m (2 ft.) of joint/crack edge. Problem Loose debris on the pavement, roughness, generally an indicator of advanced joint/crack deterioration Possible Causes

Possible causes are:

Excessive stresses at the joint/crack caused by infiltration of incompressible materials and subsequent expansion (can also cause blowups).

- Disintegration of the PCC from freeze-thaw action or “D” cracking.
- Weak PCC at a joint caused by inadequate consolidation during construction. This can sometimes occur at a construction joint if

  (1) low quality PCC is used to fill in the last bit of slab volume or

  (2) dowels are improperly inserted.

- Misalignment or corroded dowel.
- Heavy traffic loading.

Repair

Spalling less than 75 mm (3 inches) from the crack face can generally be repaired with a partial-depth patch. Spalling greater than about 75 mm (3 inches) from the crack face may indicated possible spalling at the joint bottom and should be repaired with a full-depth patch.

2. Faulting A difference in elevation across a joint or crack usually associated with undoweled JPCP. Usually the approach slab is higher than the leave slab due to pumping, the most common faulting mechanism. Faulting is noticeable when the average faulting in the pavement section reaches about 2.5 mm (0.1 inch). When the average faulting reaches 4 mm (0.15 in), diamond grinding or other rehabilitation measures should be considered

Problem: Roughness

Possible Causes: Most commonly, faulting is a result of slab pumping. Faulting can also be caused by slab settlement, curling and warping. Repair Faulting heights of less than 3 mm (0.125 inch) need not be repaired. Faulting in an undoweled JPCP between 3 mm (0.125 inch) and 12.5 mm (0.5 inch) is a candidate for a dowel bar retrofit. Faulting in excess of 12.5 mm (0.5 inches) generally warrants total reconstruction.

3.Polished Aggregate

Problem: Decreased skid resistance

Possible Causes: Repeated traffic applications. Generally, as a pavement ages the protruding rough, angular particles become polished. This can occur quicker if the aggregate is susceptible to abrasion or subject to excessive studded tire wear.

Repair

- HMA: Apply a skid-resistant slurry seal or BST or overlay.
- PCC: Diamond grinding or overlay.

4.Shrinkage Cracking Hairline cracks formed during PCC setting and curing that are not located at joints. Usually, they do not extend through the entire depth of the slab. Shrinkage cracks are considered a distress if they occur in an uncontrolled manner.

Problems: Aesthetics, indication of uncontrolled slab shrinkage. In JPCP they will eventually widen and allow moisture infiltration. In CRCP, if they are allowed to get much wider than about 0.5 mm (0.02 inches) they can allow moisture infiltration (CRSI, 1996).

Possible Causes: All PCC will shrink as it sets and cures, therefore shrinkage cracks are expected in rigid pavement and provisions for their control are made. However, uncontrolled shrinkage cracking can indicate:

- Contraction joints sawed too late. In JPCP, if contraction joints are sawed too late the PCC may already have cracked in an undesirable location.
- Poor reinforcing steel design. In CRCP, proper reinforcing steel design should result in shrinkage cracks every 1.2 – 3 m (4 – 10 ft.).
- Improper curing technique. If the slab surface is allowed to dry too quickly, it will shrink too quickly and crack.
- High early strength PCC. In an effort to quickly open a newly constructed or rehabilitated section to traffic, high early-strength PCC may be used. This type of PCC can have a high heat of hydration and shrinks more quickly and to a greater extent than typical PCC made from unmodified Type 1 portland cement.

Repair In mild to moderate severity situations, the shrinkage cracks can be sealed and the slab should perform adequately. In severe situations, the entire slab may need replacement.

5.Pumping Movement of material underneath the slab or ejection of material from underneath the slab as a result of water pressure. Water accumulated underneath a PCC slab will pressurize when the slab deflects under load. This pressurized water can do one of the following:

- Move about under the slab.
- Move from underneath one slab to underneath an adjacent slab. This type of movement leads to faulting.
- Move out from underneath the slab to the pavement surface. This results in a slow removal of base, subbase and/or subgrade material from underneath the slab resulting in decreased structural support.

Problem

Decreased structural support of the slab, which can lead to linear cracking, corner breaks and faulting.

Possible Causes: Water accumulation underneath the slab. This can be caused by such things as: a high water table, poor drainage, and panel cracks or poor joint seals that allow water to infiltrate the underlying material.

Repair: First, the pumping area should be repaired with a full depth patch to remove any deteriorated slab areas. Second, consideration should be given to using dowel bars to increase load transfer across any significant transverse joints created by the repair. Third, consideration should be given to stabilizing any slabs adjacent to the pumping area as significant amounts of their underlying base,subbase or subgrade may have been removed by the pumping. Finally, the source of water or cause of poor drainage should be addressed.

6.Punchout Localized slab portion broken into several pieces Problem Roughness, allows moisture infiltration leading to erosion of base/subbase support, cracks will spall and disintegrate.

Possible Causes: Can indicate a localized construction defect such as inadequate consolidation. In CRCP, it can be caused by steel corrosion, inadequate amount of steel, excessively wide shrinkage cracks or excessively close shrinkage cracks.

Repair: Full-depth patch.

Linear Cracking Linear cracks not associated with corner breaks or blowups that extend across the entire slab. Typically, these cracks divide an individual slab into two to four pieces. Often referred to as “panel cracking”

Problem: Roughness, allows moisture infiltration leading to erosion of base/subbase support, cracks will eventually spall and disintegrate if not sealed

Possible Causes: Usually a combination of traffic loading, thermal gradient curling, moisture stresses and loss of support.

Repair: Slabs with a single, narrow linear crack may be repaired by crack sealing. More than one linear crack generally warrants a full-depth patch

7.Joint Load Transfer System Deterioration Transverse crack or corner break developed as a result of joint dowels. Problem Indicator of a failed load transfer system, roughness

Possible Causes Load transfer dowel bars can fail for two principal reasons:

- Corrosion. If inadequately protected, dowel bars can corrode over time. The corrosion products occupy volume, which creates tensile stresses around the dowel bars, and a severely corroded dowel bar is weaker and may fail after repeated loading.
- Misalignment. Dowel bars inserted crooked or too close to the slab edge may create localized stresses high enough to break the slab. Misalignment can occur during original construction or during dowel bar retrofits. Repair Removal and replacement of the affected joint load transfer system followed by a full-depth patch for affected area.

Repair: Removal and replacement of the affected joint load transfer system followed by a full-depth patch for affected area.

8.Blowup: A localized upward slab movement and shattering at a joint or crack. Usually occurs in spring or summer and is the result of insufficient room for slab expansion during hot weather.

Problem: Roughness, moisture infiltration, in extreme cases (as in the second photo) can pose a safety hazard

Possible Causes: During cold periods (e.g., winter) PCC slabs contract leaving wider joint openings. If these openings become filled with incompressible material (such as rocks or soil), subsequent PCC slab expansion during hot periods (e.g., spring, summer) may cause high compressive stresses. If these stresses are great enough, the slabs may buckle and shatter to relieve the stresses. Blowup can be accelerated by:

- Joint spalling (reduces slab contact area and provides incompressible material to fill the joint/crack
- D cracking (weakens the slab near the joint/crack area)
- Freeze-thaw damage (weakens the slab near the joint/crack area)

Repair: Full-depth patch

9. Popouts: Small pieces of PCC that break loose from the surface leaving small divots or pock marks. Popouts range from 25 – 100 mm (1 – 4 inches) in diameter and from 25 – 50 mm (1 – 2 inches) deep.

Problem Roughness, usually an indicator of poor material

Possible Causes: Popouts usually occur as a result of poor aggregate durability. Poor durability can be a result of a number of items such as:

- Poor aggregate freeze-thaw resistance
- Expansive aggregates
- Alkali-aggregate reactions

Repair: Isolated low severity popouts may not warrant repair. Larger popouts or a group of popouts can generally be repaired with a partial depth patch.

Corner Break

Problem: Roughness, moisture infiltration, severe corner breaks will fault, spall and disintegrate **Possible Causes:** Severe corner stresses caused by load repetitions combined with a loss of support, poor load transfer across the joint, curling stresses and warping stresses.

Repair: Full-depth patch

CBR METHOD OF FLEXIBLE PAVEMENT:

7.5.2 Principle of CBR Method of Pavement Design

Development of the CBR method

In 1928 California Division of Highways in the U.S.A. developed California Bearing Ratio (CBR) method for pavement design. The CBR tests were carried out by the California State Highway Department on existing pavement layers including subgrade, sub-base and base course. Based on the extensive CBR test data collected on pavements which behaved satisfactorily and those which failed, an empirical design chart was developed correlating the CBR value and the pavement thickness. The basis of the design chart is that a subgrade soil with a given CBR value required a certain thickness of flexible pavement as a cover.

Basic principle

The details of CBR tests are given in Chapter – 6, Highway Materials. The basic principle of CBR method of flexible pavement design is based on the concept that the total thickness of flexible pavement required mainly depends upon two factors, namely (i) CBR value of the soil subgrade over which the pavement is to be laid and (ii) the magnitude of the wheel load or intensity of traffic loads expected.

A weaker soil subgrade with lower CBR value will need a flexible pavement of higher thickness. Also a subgrade soil with any particular CBR value will require thicker flexible pavement structure to cater for higher magnitude of design wheel load or for higher intensity of traffic loads.

Advantages of CBR method

One of the main advantages is the simplicity of conducting the CBR test in the laboratory as well as the method of pavement design using simple design charts. The CBR method of flexible pavement design was being extensively used in different countries of the world for quite a long period of time. However based on the local design requirements (such as traffic, climatic and other environmental factors) each country developed their own design chart. For example the United Kingdom (UK) and several states in the USA, developed their own empirical pavement design charts making use of the CBR value of the subgrade soil.

Limitations of CBR method

There are several limitations of both the CBR test and the design method. Some of these are given below.

(a) It is important to understand the limitations of the CBR test itself on subgrade soil, which is an empirical penetration test for assessing the strength characteristics. The CBR value does not represent any of the basic strength properties of the soil

(b) The punching shear under the CBR test condition does not in any way represent the stress on the subgrade through the flexible pavement layers due to traffic wheel loads

(c) It is necessary to judiciously decide the soaking requirements or the testing moisture content while determining the CBR value of a subgrade soil. For example in an arid region with very scanty rainfall and if the subsurface water level is very deep below the ground level ...

(d) The specified four days soaking period may not be sufficient in some highly clayey soils to represent the worst field moisture content in areas with water-logging

(e) The general limitations of empirical pavement design method as given in Art. 7.5.1 above are applicable in this case also

(f) The total thickness of flexible pavement designed by the CBR method depends only on the CBR value of the subgrade soil. The total thickness remains the same irrespective of the type of materials used in different pavement layers

(g) The CBR design charts developed in a certain region or country based on performance studies represent the other design factors such as traffic loads/intensity, materials used in different pavement layers, climatic, drainage and other environmental factors pertaining to that region. The same design chart may not be suitable in another region with different set of design factors

7.5.3 Pavement Thickness Determination

In order to design a pavement by CBR method, first the soaked CBR value of the soil subgrade, C % is determined in the laboratory. Then the appropriate design curve is chosen from the design chart, depending on the design wheel load or the design traffic. The total thickness of flexible pavement, say T mm needed over the subgrade of CBR value, C % is obtained directly from the design chart. In case another material superior than the soil subgrade with CBR value C_1 % is available for construction of a sub-base course, then the thickness of construction T_1 mm over the sub-base course may be obtained by using the same design curve (for the desired wheel load or traffic intensity). As the CBR value C_1 % of the sub-base material is higher than that of subgrade soil, the thickness T_1 mm required above the sub-base will be lower than the total thickness T mm. The thickness of the granular sub-base course is equal to the total thickness minus the thickness over the sub-base material i.e., $(T - T_1)$ mm.

7.5.4 Precautions during CBR Test and Design Method

(a) The CBR tests should be performed on remoulded soils in the laboratory. In-situ tests are not recommended for design purposes. The specimens should be prepared by static compaction at desired density or by dynamic compaction. The standard test procedure should be strictly adhered to

(b) For the design of new roads for National and State Highways, the top 500 mm of subgrade soil sample should be compacted at OMC to the specified dry density. The thickness of the top layer of the subgrade can be 300 mm for low volume rural roads. As per the 'Specifications for Road and Bridge Works' by the MORTH, the specified density of compaction is 97 % of density by Heavy Compaction. Otherwise the soil sample may be compacted to the dry density expected to be achieved in the field. In the case of existing roads, the sample should be compacted to field density of subgrade soil (at OMC or at a field moisture content)

(c) In new constructions the CBR test samples may be soaked in water for four days period before testing. However in areas with arid climate or when the annual rainfall is less than 500 mm and the water table is too deep to affect the subgrade adversely and when thick and impermeable bituminous surfacing is

UNIT 6.HIGHWAY CONSTRUCTION AND MAINTENANCE

Unit 6 Highway Construction and Maintenance

Highway construction: Earthen roads, WBM roads, bituminous concrete roads and cement concrete roads, Highway maintenance – Maintenance of Earth roads, bituminous surfaces, Cement concrete roads.

Introduction:

The Highway construction may be taken up in two stages.

A) Earth work and preparation of sub grade

B) Pavement Structure.

The Earth work mainly consists of preparation of the sub grade to be suitable to the subsequent construction of the pavement structure.

Various types of roads that are preferred to construct in Highways are

1) Earthen roads & Gravel roads,

2) Soil stabilized roads,

3) Water bound macadam (WBM) roads,

4) Bituminous or Black top roads,

5) Cement Concrete roads etc.

CONSTRUCTION OF EARTH ROADS:

These are considered to be the cheapest roads as they are prepared from the natural soil. The pavement section is totally made out of the soil available at site and at nearby borrows pits. The type of construction largely depends upon the type of the soil at site. The camber provided to the earth roads is very steep and ranges between 1 in 20 to 1 in 33.The steep cross slope also helps in keeping the pavement surface free of standing water, otherwise the soil being pervious the water would damage the pavement section by softening it. The maximum cross slope of 1 in 20 is recommended to avoid erosion due to rain waters and formation of cross ruts.

	Base course	Wearing Course
Clay	< 5%	10 to 18%
Silt	9 to 32%	5 to 15%
Sand	60 to 80%	65 to 80%
Liquid limit	<35%	< 35%
Plasticity index	<6%	4 to 10%

The construction procedure is as follows:

i) Material: The soil survey has to be carried out and suitable borrow pits are located within economical haulage distances. The borrow pits are usually selected outside the land width. The trees, shrubs, grass roots and other organic matter including the top soil are removed before excavating earth for construction.

ii) Location: The center line of road edges are marked on the ground along the alignment by driving the wooden pegs. Reference pegs are also driven to help in following the desired vertical profile of the road during construction. The spacing of the reference pegs depends on the estimated length of road construction per day.

iii) Preparation of Sub Grade: The various operations involved in the preparation of the sub grade are as follows:

a) Clearing the site

b) Excavating and construction of fills to bring the road to a desired grade

c) Shaping of sub grade the site clearance may be carried out manually using the appliances like spade, pick and hand shovel. Mechanical equipment like dozer, scraper and ripper may also be used for the purpose.

The subgrade should be graded to the desired camber and longitudinal profile and to the desired depth depending on the thickness of the pavement construction it is desirable to compact the subgrade before placing the pavement layers.

iv) Pavement Construction: The borrowed soil is dumped on the prepared sub grade and pulverized. The field moisture content is checked and additional water content is added if necessary to bring it up to OMC. The soil is mixed, spread and rolled in layers such that the compacted thickness of each layer does not exceed 10 cm. The type of roller for compaction is decided based on soil type, desired amount of compaction and availability of equipment. At least 95 % of dry density of I.S light compaction is considered desirable. The camber of the finished pavement surface is checked and corrected if necessary.

v) Opening to Traffic: The compacted earth road is allowed to dry out for a few days before opening it to traffic.

Maintenance of Earth Roads:

The usual damages caused in the earth roads needing frequent maintenance are

(i) Formation of dust in dry weather
(ii) Formation of longitudinal ruts along wheel path or vehicles
(iii) Formation of cross ruts along the surface after monsoons due to surface water.

Thus, dust nuisance may be remedied by the following methods:

Frequent sprinkling of water

Treatment with calcium chloride

Use of other dust palliatives

Application of calcium chloride retains some water due to hygroscopic nature of the mix. Oiled roads earth roads are also common these days.

•Periodical maintenance by spreading moist soil along ruts and reshaping of camber is necessary. Formation of cross ruts may be due to excessive cross slope. However in the areas of heavy rain fall, it may not be possible to avoid these in untreated earth road. Hence either these ruts should be repaired from time to time during and after the monsoon or a surface treatment or stabilized layer be provided on the top.

Advantages:

- The construction of earth road is a fast process.
- Proper selection of the gradient give balanced earthwork.
- In future if other type of road is going to be constructed on the existing earth road, it gives good foundation.
- The overall process is relatively cheaper than other road types.

Disadvantages:

- These roads are only useful for light traffic. It cannot sustain the lifespan of the road if it is allowed for heavy traffic.
- This type of road wears quickly and the maintenance is little bit costlier.
- This type of road cannot be constructed or it will be worthless in the areas where monsoon is on peak or areas that have maximum rainfall, as constant and excess rainfall lashes out these kinds of roads.

Water Bound Macadam:

The water bound macadam (WBM) is the construction known after the name of John macadam. The term macadam in the present day means the pavement base course made of crushed and broken aggregates mechanically interlock by Rolling and the voids filled with screening and binding material with the assistance of water. The WBM may be used as a sub base course or surface course the thickness of each compacted layer of WBM ranges from 10 to 7.5 CM depending on the size and gradation of the aggregates used the number of layers and the total thickness of WBM construction depends on the design details of the payment.

When used as surface course WBM get deteriorated rapidly under adverse conditions of traffic and weather therefore it is desirable to provide a bituminous surface course over the WBM layer in order to prolong its life.

Specifications of materials for WBM pavement

1. **Type of course aggregates:** Over burnt brick metals, naturally occurring soft aggregates such as kankar or laterite may be used. Crushed slag obtained from blast furnace may also be used.

Property	Requirements for pavement layer		
	Sub-base	Base course	Surfacing course
Los Angeles abrasion value (maximum value, percent)	60	50	40
Aggregate impact value (maximum value, percent)	50	40	30
Flakiness index (maximum value, percent)	-	15	15

2. **Properties of course aggregates** Crushed stone aggregates should be hard, durable and of acceptable shape, free from flaky and elongated particles.
3. **Size and Grading requirements of course aggregates:**
   Grading no.1 is more suitable for sub-base course. Thickness of compacted layer is 100mm
   • Compacted thickness of each layer for grading no.2 and grading no.3 is 75mm.

Grading No.	Size range, mm	Sieve size, mm	Percent passing the sieve, by weight
1	90 to 40	100	100
		80	65 – 85
		63	25 – 60
		40	0 – 15
		20	0 – 5
2	63 to 40	80	100
		63	90 – 100
		50	30 – 70
		40	0 – 15
		20	0 – 5
3	50 to 20	63	100
		50	95 – 100
		40	35 – 70
		20	0 – 10
		10	0 – 5

Screenings The screenings consists of smaller size, generally of the same material as the course aggregates.

Classification grading	Size of screenings mm	Sieve size, mm	Percent passing the sieve, by weight
A	12.5	12.5	100
		10.0	90 – 100
		4.75	10 – 30
		0.15	0 – 8
B	10.0	10.0	100
		4.75	85 – 100
		0.15	10 – 30

• **IRC** has suggested that non- plastic materials such as kankar nodules, moorum or gravel other (than river borne rounded aggregate) may be used as screening materials, provides Liquid limit is less than 20%, Plasticity index is less than 6% and portion of fines passing 0.075mm sieve is less than 10%.

Binding material

• Binder material consists of fine grained material is used in WBM construction.

• The plasticity index value of WBM,

 • For surface course, **4-9%**

 • For base or sub-base course with bituminous surfacing, **6%**

CONSTRUCTION PROCEDURE:

1. Preparation of Foundation for Receiving the WBM Course

• The foundation for receiving the new layer of WBM may be either the subgrade or sub-base or base course.

• This foundation layer is prepared to the required grade and camber and the dust and either loose materials are cleaned.

On existing road surfaces, the depressions and pot-holes are filled and the corrugations are removed by scarifying and reshaping the surface to the required grade and camber as necessary.

• If the existing surface is a bituminous surfacing, furrows of depth 50 mm and width 50 mm are cut at 1 m intervals and at 45 degrees to the center line of the carriageway before laying the coarse aggregate.

Figure : site clearance

2. Provision of Lateral Confinement:

Lateral confinement is to be provided before starting WBM construction. This may be done by constructing the shoulders to advance, to a thickness equal to that of compacted WBM layer and by trimming the inner sides vertically.

3. Spreading of course aggregates:

• The coarse aggregates are spread uniformly to proper profile to even thickness upon the prepared foundation and checked by templates.

• WBM course is normally constructed to compacted thickness of 7.5 cm except when using grade 1 it is 10 cm.

4. Rolling:

After spreading the coarse aggregates properly, compaction is done by a three wheeled power roller of capacity 6 to 10 tones.

Rolling is started from the edges, the roller being run forward and backward until the edges are compacted. The run of the roller is then gradually shifted towards the center line of the road. Uniformly overlapping each preceding rear wheel track by one half width. This process is repeated by rolling from either edge towards the center line until adequate compaction is achieved.

5. Application of Screenings:

After the coarse aggregates arc rolled adequately, the dry screenings are applied gradually over the surface to fill the interstices in three or more applications. Dry rolling is continued as the screenings are being spread and brooming carried out.

Fig: Screenings

6. Sprinkling and Grouting:

After the application of screenings, the surface is sprinkled with water, swept and rolled. Wet screenings are swept into the voids using hand brooms. Additional screenings are applied and rolled till the coarse aggregates are well bonded and firmly set.

7. Application of Binding material:

After the application of screening and rolling, binding material is applied at a uniform and slow rate at two or more successive thin layers. After each application of binding material, the surface is copiously sprinkled with water and wet slurry swept with broom to fill the voids. This is followed by rolling with a 6 to 10 tonnes roller and water is applied to the wheels to wash down the binding materials that stick to the roller. When crushable type screenings like moorum or gravel are used, there is no need to apply binding materials, except in the surfacing course.

Fig: Moorum binding material

8. Setting and Drying:

After final compaction, the WBM course is allowed to set over-night. On the next day the 'hungry' spots are located and arc filled with screenings or binding material, lightly sprinkled with water if necessary and tolled. No traffic is allowed till the WBM layer sets and dries out. In the case of WBM base course, the layer is allowed to dry completely without permitting traffic to ply and then the bituminous surfacing is laid. Limited construction traffic may be permitted to ply over the WBM layer taking Proper care not to damage the layer.

Summary of the procedure of WBM road Construction:
Prepared subgrade + Spreading of Coarse aggregates + Compaction (Rolling) + application of screening material (Gravel or stone dust) + Rolling + Application of Binding material (Moorum)+ Rolling(Drying and open to traffic) = WBM

Maintenance of WBM Roads:

WBM is the basic stage of the planned improvement of road surfacing. Wbm roads are damaged rapidly due to the heavy traffic and adverse climatic conditions. The steel tyred bullock carts cause serve wear and tear to the WBM surface. In dry weather, dust is formed and in rainy season mud is formed on WBM road.

Fine particles are sucked up by fast-moving vehicle from the road surface resulting in loss of binding particles in the surface. In the rainy season, the rain is soaked by the surface which makes the surface soft. In such conditions, the movement of vehicles makes the layer of surface loose. In this situation, some aggregate gets displaced from their position causing ruts and potholes on the surface.

- Whenever the potholes and ruts occur on the road by the period of time, they should be filled with adequate materials and proper compacting should be done.
- The corrugations occurred on the roads should be removed by means of dragging. If not could make the condition worse.
- Broken materials of the roads should be properly restored by fresh materials.
- The surface of the road should be renewed in 2-5 years or based according to the traffic volume.
- The loose aggregates starts coming on the top of the surface of the road, they should be removed and leveled surface should be added by fresh binding material and it should be properly watered and compacted.
- To prevent aggregate from getting loose from the surface course, a thin layer of moist soil should be spread over the surface periodically, particularly after the rainy season.
- The dust nuisance can be effectively eliminated by providing a surface dressing of bituminous materials.

Advantages of WBM Road:

- The construction cost of WBM road is comparatively low.
- In the construction of WBM road no skilled labors are required.
- They are constructed from locally available materials.
- If the WBM roads are maintained properly and from time to time, it can resist load of traffic of about 900 tonnes per lane per day.

Disadvantages of WBM Road:

- The maintenance cost of WBM roads is high.
- The overall life span of these roads is very less.
- If the WBM roads are not properly maintained they can cause inconvenience and danger to traffic.
- As WBM roads are permeable to rain water, it leads to softening and yielding of subsoil.

Construction of Bituminous Concrete Pavement:

- The bituminous concrete is the highest quality of construction in the group of black top surfaces
- Being of high cost specifications, the bituminous mixes are properly designed to satisfy the design requirements of the stability and durability.
- The mixture contains dense grading of coarse aggregates fine aggregates and mineral filler coated with bituminous Binder the mix is prepared in hot mix plant.

The thickness of the bituminous concrete layer depends upon the traffic and quality of the base course the specifications of materials and construction steps of the bituminous concrete or Asphalt concrete (AC) surface course or given below.

CONSTRUCTION OF FLEXIBLE PAVEMENTS

Components of Flexible Pavements:

The component layers of a flexible pavement laid over the subgrade are:

(a) Granular sub-base and drainage layer

(b) Granular base course

(c) Bituminous binder course

(d) Bituminous surface course

On low-volume roads or village roads the typical components of flexible pavement may consist of the following layers:

(a) Granular sub-base or drainage layer

(b) Granular base course

(c) Thin bituminous surfacing

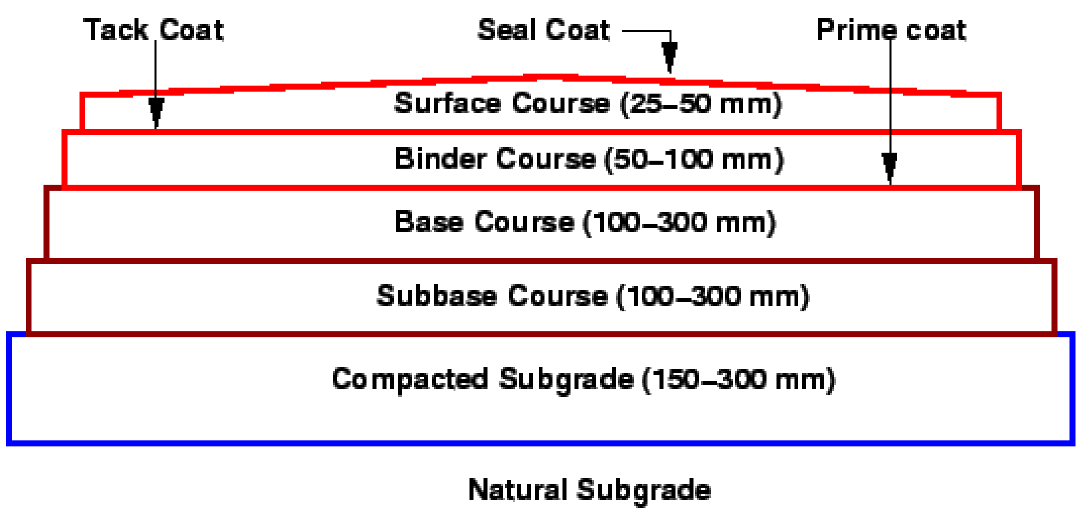


Figure . Typical cross section of a flexible pavement

Construction of Granular Sub-base Course/Drainage Layer:

Objects:

A granular sub-base (GSB) course is laid in between the subgrade and the base course of all highway pavements, in one or more layers. The GSB layer should be laid over the full width of the prepared subgrade, extending up to the side drains so as to serve as a 'drainage layer' of the pavement, if another separate drainage layer is not provided.

In flexible pavements of highways, the GSB layer performs the following two important **functions:**

1. To serve as an effective drainage layer to drain off the rain water entering into the pavement layers.
2. To serve as a structural component of the flexible pavement structure by distributing the wheel loads.

Materials for GSB layer:

The materials used for the construction of the GSB layers are,

(i) Crushed stone aggregates
(ii) Gravel
(iii) Coarse sand and
(iv) Selected soils such as moorum with less fines and very low plasticity.

Different grading of GSB materials have been suggested in the MORTH Specifications.

The specified requirements of material used for GSB layer are as under:

(a) Passing 0.425 mm sieve shall have liquid limit less than 25 percent and plasticity index less than 6.0 percent

(b) Fines passing 0.075 mm sieve, less than 10 percent

(c) CBR value not less than 30 percent for important highways with heavy traffic and not less than 25 or 20 percent in other cases.

Construction method:

The GSB layer is constructed on the top of the prepared subgrade; therefore first the surface of the subgrade in checked and grass and vegetation if any, are removed. The grade and Cross slope of the top surface of the subgrade are corrected as required.

The construction steps are given below:

(i) The sub-base material is spread to uniform thickness and specified cross slope using a motor grader by adjusting the blade of the grader

(ii) The moisture content of the material is checked and the additional quantity of water required to bring up to the optimum moisture content is sprinkled at an uniform rate using a truck mounted sprinkler

(iii) The watered Material is mixed properly using machinery such as disc harrows and rotavators.

(iv) The mixed material is spread to the desired thickness, grade and camber using a motor grader with hydraulic controls of the blade

(v) The loose GSB layer is compacted by rolling; if the compacted thickness of the layer is 100 mm or lesser, an ordinary smooth wheeled roller may be used; for compacted thickness exceeding 100 mm and up to 225 nun, compaction is clone by vibratory roller of static weight 10 tonnes or more; pneumatic tyred roller of appropriate gross load and tyre pressure may also be used if the type of material selected can be effectively compacted by this equipment.

(vi) Rolling is done starting from the lower edge and proceeded towards the center of the undivided carriageway or towards the upper edge of the divided carriageway, with a minimum one third overlap between each run of the roller; the rolling speed is limited to less than 5 kmph.

(vii) Rolling la continued till at least 98 percent of maximum density of the material is achieved.

Construction of Granular Base Course:

- Types of base course materials used in flexible pavements the common types of base course materials used in India for the construction of flexible pavements are **'Wet Mix Macadam'** (WMM), **'Water Bound Macadam'** (WBM), **Soil-aggregate mixes and stabilized soil mixes**.
- The flexible pavements of most of the important highways are being constructed , These days using WMM as the base course.
- There are various other types of base course materials that can be used in the base course or lower layer of base course of flexible pavements; these include 'Crusher-run-Macdam' and also different types of aggregates, soil-aggregate mixes or soils stabilized using different types of stabilizers.

Material:

Wet mix macadam base course consists of a well graded hard aggregates and adequate portion of the water mixed thoroughly in a mix plant.

Aggregates fulfilling the following physical properties are used;

Los Angeles abrasion value : less than 40 percent,

or

Aggregate impact value : less than 30 percent

Combined flakiness and elongation index : less than 30 percent

Plasticity index of material finer than 0.425 mm sieve : less than 6.0

Two different grading requirements have been suggested for use in WMM layers as given in Table 8.1, to be adopted depending on the thickness of each compacted layer.

Table 8.1 Grading requirement of aggregates for wet mix macadam

Sieve size, mm	Passing the sieve, percent by weight	
	Grading – 1	Grading – 2
53.00	100	-
45.00	95 to 100	-
26.50	-	100
22.40	60 to 80	50 to 100
11.20	40 to 60	-
4.75	25 to 40	35 to 55
2.36	15 to 30	-
0.06	8 to 22	10 to 30
0.075	0 to 8	2 to 9

Construction steps:

(i) Compaction test is carried out in the laboratory using the selected grade of WMM material, after removing the fraction of aggregates retained on i9 mm sieve and replacing it with material passing 19mm sieve and retained on 4.75 mm sieve. The optimum moisture content of the WMM mix is, determined in the laboratory under heavy compaction.

(ii) The selected WMM mix (with water equal to the optimum moisture content added) is prepared in a suitable mixing plant like the 'pug mill'

(iii) The WMM mix is transported to the site and is spread using a self-propelled type paver-finisher machine, to the required thickness, grade and cross

(iv) The WMM layer is compacted using a vibratory roller of minimum weight of 10 tonnes, the compacted thickness of each layer should be less than 200 mm.

Rolling is done starting from the lower edge and proceeded towards the centre of the undivided carriageway or towards the upper edge of the undivided carriageway, with a minimum overlap between each run of the roller: the rolling speed is limited to less than 5 kmph.

(v) If the total design thickness of WMM base course is say 250 mm, the base is constructed in two layers, each of compacted thickness 125 mm. After compaction of the first layer, the second layer is laid by a mechanical paver-finisher (preferably by a sensor-paver) and compacted by a vibratory roller as mentioned in steps (iii) and (iv) above

(vi) The WMM surface is checked for defects, if any and allowed to dry; no traffic shall be allowed before a bituminous surface course is constructed.

(vii) After the WMM layer is dried for at least 24 hours in dry weather, the preparation for laying a bituminous pavement layer may start by applying the prime coat.

Bituminous Surface Course:

Different type of bituminous layers are being used as surface course of flexible pavements
Thin bituminous surfacing is provided on roads with light traffic.
Thicker bituminous layers are needed to withstand heavier traffic loads, additional bituminous pavement layers in the form of binder course or base course and binder course are laid before laying the bituminous surface course.

It is essential to provide an appropriate type of ***'interface treatment'*** before laying any type of bituminous layer over another layer. If the bituminous layer is to be laid over a granular or non-bituminous base or sub-base course, consists of application of both 'prime coat' and 'tack coal'.
If the bituminous layer is to he laid over on existing bituminous surface, the interface treatment consists of application of only tack coat.
Different types of bituminous base course that have been used in India are:

i) Bituminous Macadam
ii) Penetration Macadam and
iii) Built-up Spray Grout.

Different types of bituminous binder course that have been in use are: it,

(i) Bituminous Macadam and

(ii) Dense Bituminous Macadam.

Different types of thin bituminous surface course that are being used in this country on roads with low to moderate traffic volume are

(i) Bituminous Surface Dressing
(ii) Open-graded Premix Carpet with Seal Coat
(iii) Close Graded Premix Surfacing or Mixed Seal Surfacing.

Though 25 mm Semi-dense Bituminous Concrete and 25 mm bituminous concrete are thin surface course layers, they are generally laid either over existing bituminous surface for resurfacing or over a bituminous base course.

These thin dense graded bituminous layers shall not be directly laid over granular base course. Thick layers of high quality bituminous surfacing being laid in India generally consist of 40 or 50 mm layer of ***'Bituminous Concrete'*** laid over one or more layers of 'Dense Bituminous Macadam' binder course, particularly on important roads carrying heavy wheel loads.

The following types of interfaces are used:

Seal Coat:

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

Tack Coat:

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

Prime Coat:

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

Dense graded bituminous mix:

Dense graded bituminous mixes are prepared using well graded aggregates, suitable filler material and appropriate type, grade and proportion of bitumen binder.
Most commonly used dense graded, stiff bituminous pavements layers in this country are ‘Dense Bituminous Macadam’ binder course and ‘Bituminous Concrete’ surface course.

The IRC has suggested the use of dense graded bituminous mixes in the following three type’s flexible pavement layers of highways with heavy to very heavy traffic.
(a) Dense Bituminous Macadam (DBM) to be used as a strong binder course or as overlay along with suitable dense graded surface course for strengthening existing pavement; the DBM layers are to be laid as single or in multiple layers, each of thicknesses 50 to 100 mm,,
(b) Bituminous Concrete (BC) surface course layer of thickness 25 or 40 or 50 mm laid in single layer.

Materials:

The materials required for the construction of dense graded bituminous mixes are:

(i) bitumen binder of appropriate type and grade,

(ii) Coarse aggregates and fine aggregates fulfilling the specified properties and gradation and

(iii) Suitable filler material.

The aggregates shall fulfill the physical requirements specified by the IRC, are given under

Aggregate impact value, or	:	less than 24 percent
Los Angles abrasion value	:	less than 30 percent
Flakiness and elongation index (combined)	:	less than 35 percent
Soundness: loss with sodium sulphate, 5 cycles, or	:	less than 12 percent
loss with magnesium sulphate, 5 cycles	:	less than 18 percent
Water absorption	:	less than 2.0 percent
Adhesion: retained bitumen coating	:	more than 95 percent
Water sensitivity: retained tensile strength	:	more than 80 percent
Polishing: Polished stone value (for surface course only)	:	more than 55

Mix design requirements

The dense graded bituminous mixes are designed by Marshall method by applying 75 blows on each face of the specimens. The designed mixes should fulfil the following mix design requirements for use in the all the three types of dense graded bituminous mixes, namely DBM binder course, SDBC surface course and BC surface course.

Marshall stability at 60°C	:	not less than 900 kg or 9.0 kN
Marshall flow value	:	2.0 to 4.0 mm
Marshall quotient (stability/flow)	:	2.0 to 5.0
Air voids	:	3.0 to 5.0 percent
Voids filled with bitumen (VFB)	:	65 to 75 percent
Water sensitivity : Tensile strength ratio	:	not less than 80 percent

Construction Steps:

(i) The receiving surface on which the dense graded bituminous mix is to be laid (such as existing bituminous surface course or bituminous base/binder count) is prepared by patching the pot-holes. scaling the cracks and filling sip the depressions; a geo-synthetic layer or stress absorbing layer may be laid if required. [Note: The DBM binder course may also be laid over a well compacted WMM base course, after applying the prime coat as specified).

(ii) If the profile correction required exceeds 40 mm, a profile corrective course is laid separately using a mechanical paver and is compacted; if the correction required is less than 40 mm, the pavement layer is spread with provision for the additional quantity of the mix to meet the profile correction requirements.

(iii) The laying of dense graded bituminous work is to be taken up during dry weather, free from dampness on the receiving surface and when the atmospheric temperature is higher than 10°C,

(iv) The receiving surface is cleaned with a mechanical broom to remove loose materials and dust and tack coat is applied as specified.

(v) The dense graded bituminous mix fulfilling the job mix formula is prepared at specified mixing temperature in an approved hot mix plant; the hot mix is transported to the construction site in insulated covered vehicles.

(vi) The mix is spread using a hydrostatic paver finisher with sensor at specified paving temperature.

(vii) Rollingling is started as soon as laying is done for short stretches; rolling is done in three stages:

(a) Initial or break down rolling using a tandem —wheel vibratory roller of dead weight 8 to 10 tonnes, set with high frequency and low amplitude of vibration

(b) Intermediate rolling using a pneumatic roller, with lyre pressure more than 5.6 kg/cm2 and of gross weight 12 to 15 tonnes; a vibratory roller may also be used if pneumatic 'Wed roller is not available and

(c) Final or finished rolling with 6 to 8 tonnes, smooth wheel roller, until roller marks are not seen on the surface: the rolling speed in all the cases shall not be more than 5.0 kmph

(viii) The compacted density achieved is checked by taking 150 mm diameter core samples; the density achieved shall preferably be 92 percent of theoretical maximum density of the mix, so that he initial voids in the max is about 7 to 8% and due to traffic induced seconder y compaction during the design life, the final voids in the mix is not lower than 4%.

(ix) The finished surface shall not be opened to traffic until the entire depth of the bituminous layer cools down to temperature below 60°C

(x) The finished surface is checked using a 3-m straight edge; the maximum permissible undulations for finished BC surface shall not exceed 5.0 mm in longitudinal profile and 4.0 mm in transverse profile. The maximum permissible number of surface unevenness of 3 to 5 mm depth under 3 rn straight edge in each 300 m stretch measured as above in the case of BC surface for national and state highways is 15

(xi) The average unevenness index or roughness index for the finished BC surface measured along the wheel path of each lane of the road using a bump integrator shall not exceed 2000 mm per km.

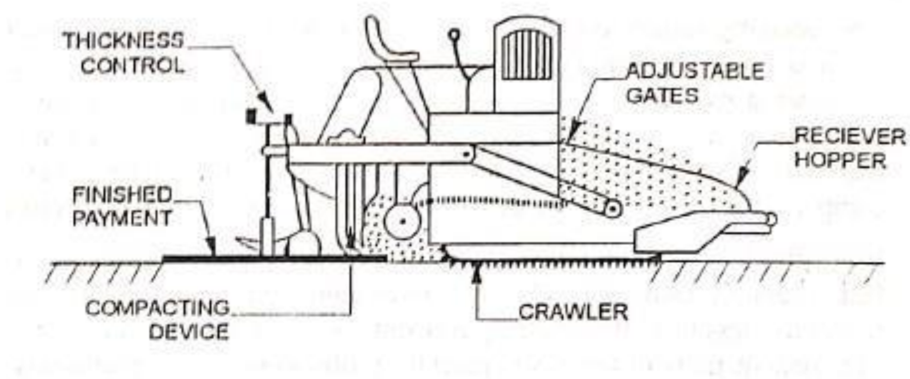


Fig. Mechanical paver finisher

CEMENT CONCRETE PAVEMENT:

Different types of cement concrete pavements

Different types of cement concrete (CC) pavements are
(i) Plain concrete pavements
(ii) Reinforced concrete pavements

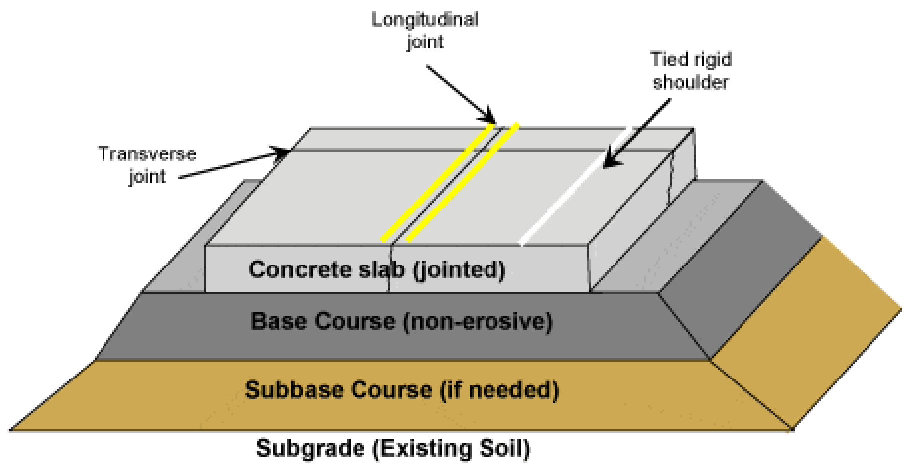
(iii) Continuously reinforced concrete pavements with elastic joints and

(iv) Fiber reinforced concrete pavements.

Of these, the plain CC pavements are the most commonly used type.

Components of Cement Concrete Pavement

The cement concrete pavement structure of major highways catering for heavy traffic loads consists of the following components, from the bottom towards the top.
(a) Soil subgrade
(b) Drainage layer
(c) Sub-base course generally constructed using lean cement concrete or 'dry lean concrete'
(d) separation membrane laid on the top of the concrete sub-base course and
(e) CC pavement slab using 'paving quality concrete' (PQC)
(f) Construction of different types of joints in CC pavements.



Construction of CC Pavement:

Construction of supporting layers

Construction of subgrade

The IRC recommends that for the construction of CC pavements of highways, the subgrade shall consist of coarse grained soil having a minimum CBR value of ***8 percent*** and be of total ***500 mm compacted thickness***. The cross slope to be provided while finishing the top of the subgrade shall be in conformity with the specified cross slope of the CC pavement. The permissible tolerance in surface levels of subgrade is (+ 20 mm and - 25 mm).

Construction of drainage layer:

The drainage layer may be directly laid over the prepared subgrade; however it keeping in view the desired long life of the CC pavement it is desirable to place a suitable type of geo-filter between the subgrade and the drainage layer.

If the water table is high, a suitable capillary cut-off may also be laid over the subgrade. The drainage layer shall extend up to the full formation width or up to the side drains.

The top surface of the drainage layer or lower sub-base shall also be in conformity with the specified cross slope of the CC pavement. The permissible tolerance in surface levels of sub-base is (+ 10 mm and -10 mm).

Construction of dry lean concrete sub-base course:

- Provision of a lean cement concrete sub-base layer below the CC pavement slab has been found to have several advantages such as providing a firm and uniform structural support, high resistance to deformation and prevention of failures due to pumping.
- 'Dry lean concrete' (DLC) is the most common type of cement treated sub-base course laid over the drainage layer. The recommended thickness of DLC sub-base course on important highways is 150 mm; a minimum thickness of 100 mm may be adopted in the case of less important roads with less traffic.
- The IRC recommends that the DLC sub-base shall extend 500 mm beyond the edges of the CC pavement in order to facilitate further construction operations and to provide good support to the CC pavement slab.
- The DLC is laid to the desired grade and cross slope with a paver. Compaction shall be carried out immediately after concrete mix is laid and is levelled by the paver.
- Double drum type smooth wheeled vibratory rollers of static weight 8 to 10 t are used for compaction of the DLC layer; the compaction is carried out to obtain 97 percent of the maximum density achieved during trial rolling.

- The surface of the DLC layer is inspected soon after completion of the final rolling; the surface deficiencies are rectified using additional concrete mix prepared using coarse aggregates of maximum size 10 mm.
- The permissible tolerance in surface levels of DLC layer is (+ 6 mm and — 15 mm).
- Curing of the DLC layer is done by spraying liquid curing compound immediately after final rolling. Further curing is continued by covering the surface with gunny bags or hessian spread in three layers and kept moist continuously for a minimum of seven days.

Lying of separation membrane

- An impermeable separation membrane made of PVC sheet of thickness 125 microns is laid between the DLC sub-base course and the CC slab. The surface of the DLC layer is thoroughly cleaned free of grit and dust.
- During hot weather, the surface of the DLC layer may be wetted with water before laying the membrane.
- The separation membrane is laid flat with minimum creases and wrinkles. When another sheet of the membrane is to be laid, the joint shall have a minimum overlap of 300 mm. The membrane may be nailed to the lower layer.

Construction of slab:

Different methods of construction of CC pavement slab:

The CC pavement slabs may be constructed by two different methods:

(i) Using ‘slip from paver’ and

(ii) Using fixed side forms and all other paving machinery.

Construction using slip form paver

- In major highway construction projects, paving equipment called `slip-form paver' is made use of.
- This equipment can perform all the functions of spreading, compacting and finishing of the CC pavement slab to the required grade and camber; additional units for texturing of the fresh concrete surface and for spraying the curing compound are also available.
- By this method of construction, there is no need to fix the side forms before laying the cement concrete mix
- During the process of laying the PQC compaction by vibrators and finishing of the CC pavement slab, temporary side supports are provided by the by the slip form paver; after completing the paving process side supports also get removed and the CC slabs remain in the position Paver moves forward, the side supports gets removed and the CC slabs

remain in the position as the CC mix is quite stiff due to low water cement ratio used in the mix.

Fig: Slip form paver

Construction using fixed side forms

Conventional side forms made of steel sections of desired depth are fixed so as to provide the required profile of the CC pavement slabs. The inside faces of the side are oiled to prevent the concrete sticking to the steel forms. In the fixed form paving method, two types of construction machinery may be used:

a) In relatively large road construction projects, a train of different fully mechanised units are used for lying spreading, compacting, finishing, texturing the surface and spraying curing compound

(b) In very small projects, when CC pavement is to be constructed in short stretches of narrow roads of less importance, semi-mechanized 'labor oriented' construction technique may be adopted; this is particularly because of the difficulty in bringing the large paving machinery within restricted road width. The side forms may be removed and shifted forward for further construction after the concrete attains sufficient strength to stand without side support.

Fig: Steel Form Work for CC pavement Construction

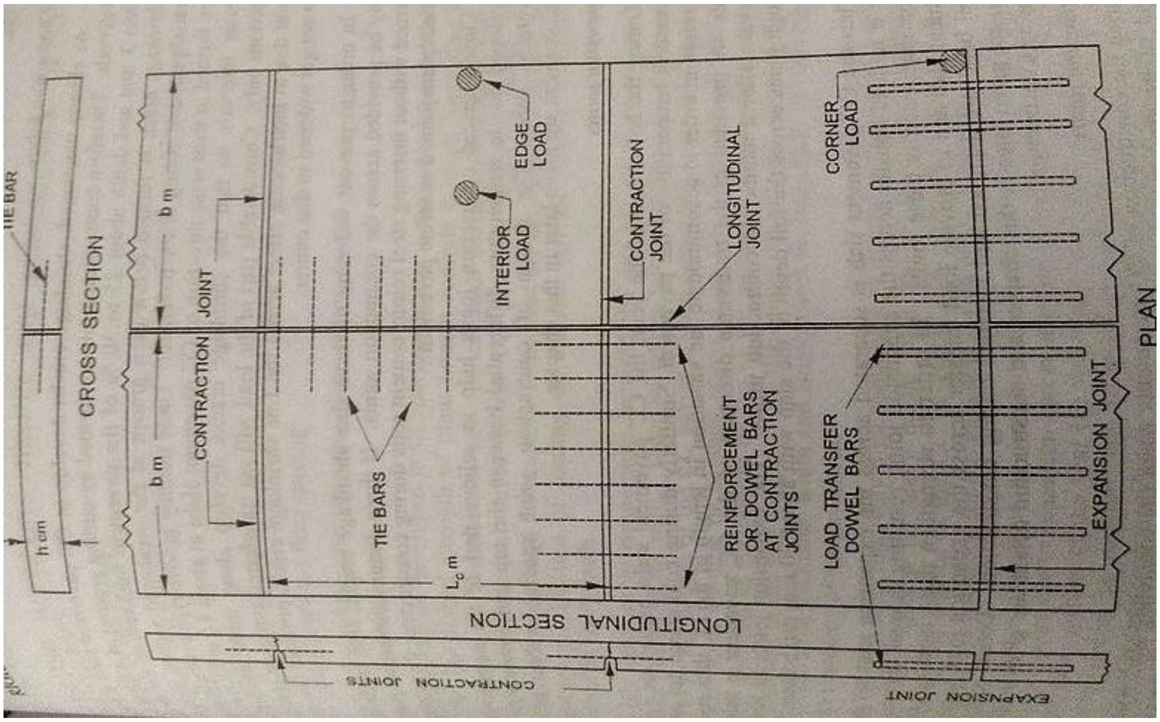
Construction of Joints in CC pavements:

Requirements of a good joint:

a) Joint must move freely,

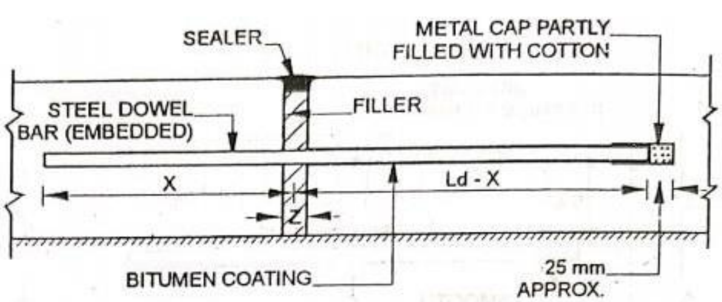
b) Joint must not allow infiltration of rain water and ingress of stone grits,

c) Joints must not protrude out of the general level of the slab.



Expansion joint: These are provided to allow for the expansion of the slabs due to the rise in slab temperature above the construction temperature of the cement concrete. These joints also permit the contraction of the slabs. These are provided at an interval of 50m - 60 m for smooth interface laid in winter and 90m-120m in summer. However for the rough interface the spacing between expansion joints may be 140m.

Transverse expansion joints are provided in CC pavements at desired intervals or at identified locations during construction of the CC pavement, with a gap of 20 to 25 mm between the slabs. Dowel bars are installed during the construction for the purpose of ***'load transfer'*** from one slab to the adjacent slab, across the expansion joint The gap is filled by compressible filler board and the top is sealed using ***sealer material***. A typical expansion joint with load transfer dowel bar is shown in Fig.



Expansion joint with dowel bar for load transfer

Components of Expansion Joint:

The expansion joints of CC pavement consist of the following components:

(a) Dowel bars

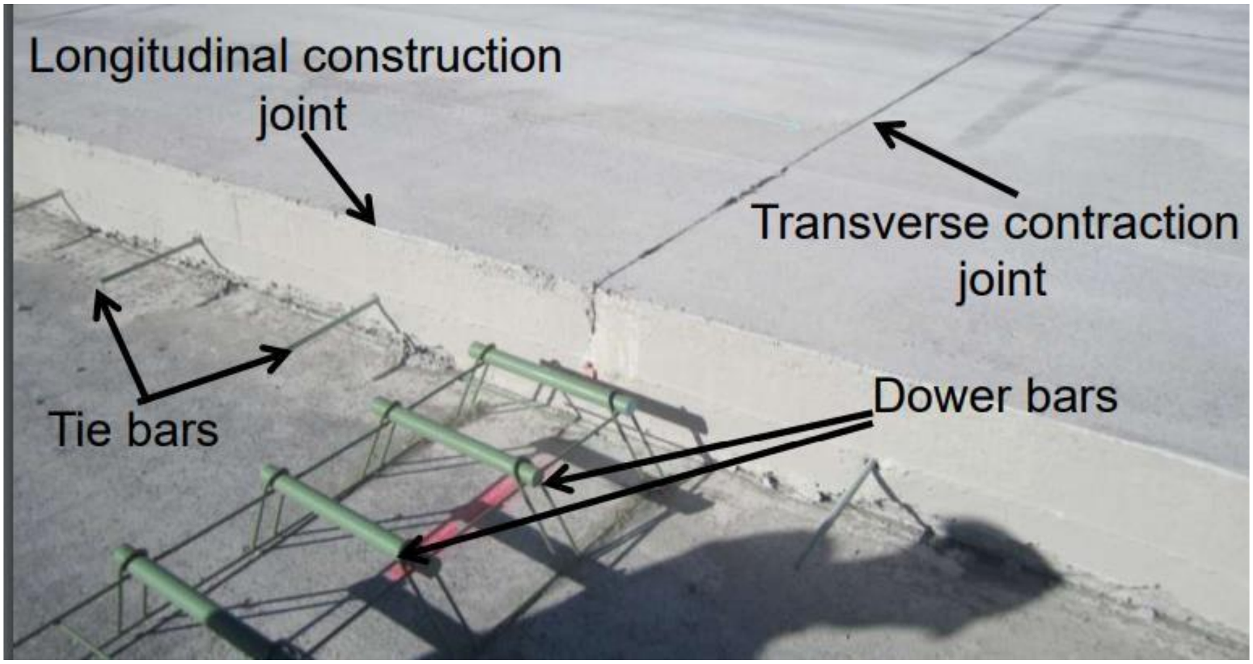
(b) Joint filler

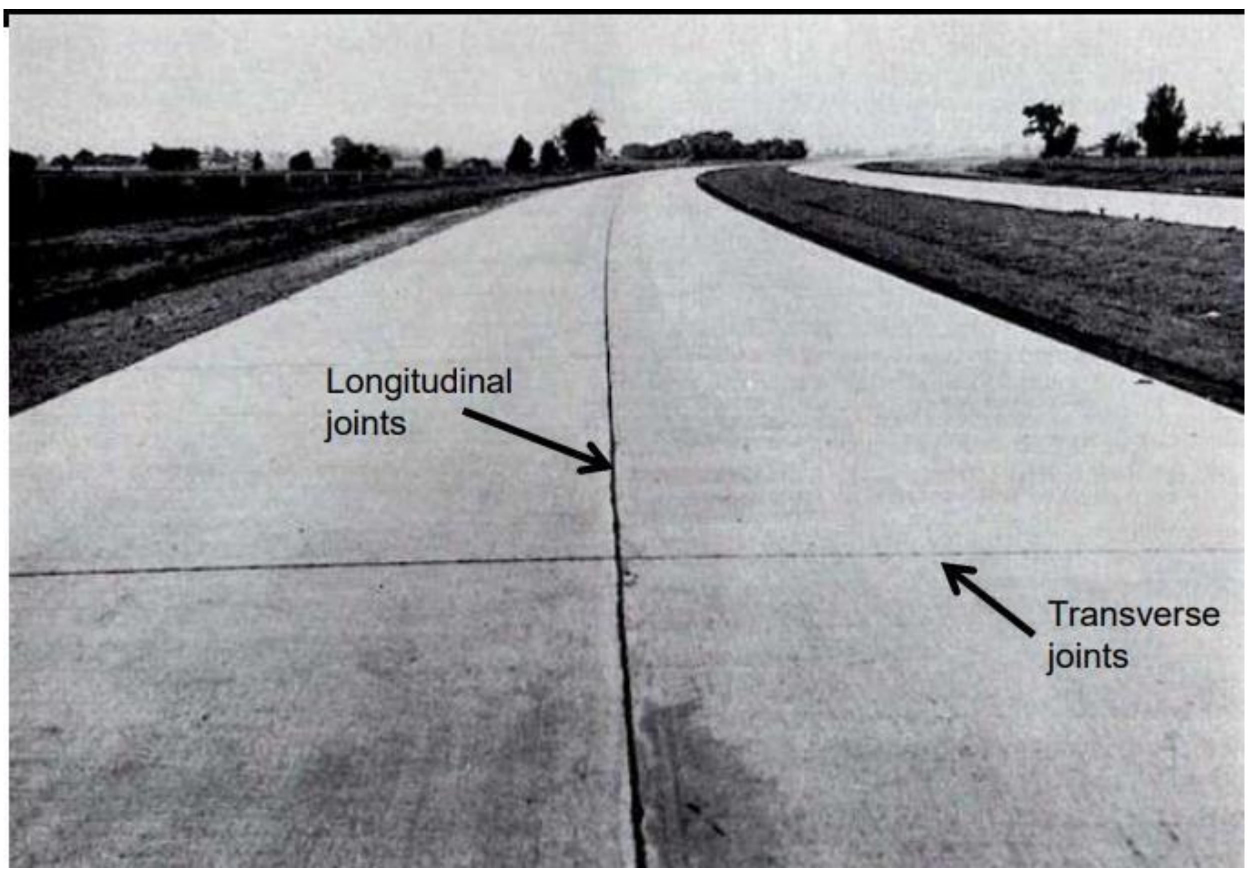
(c) Joint sealer Dowel bar

(a) Dowel Bars:

- When the wheel load approaches the edge of a CC pavement slab near the expansion joint, part of the load is expected to be transferred to the adjoining slab across the expansion joint through the dowel bar system.
- The IRC has recommended that for a design axle load of 10.2 t the diameter, length and spacing of dowel bars for slab thickness of 250 mm may be 32, 450 and 300 mm respectively; for slab thickness of 300 mm these values may be 38, 500 and 300 mm respectively.
- It is desirable to treat these dowel bars with epoxy coating or any other suitable anti-corrosion coating.

Figure Dowel bars in place at a construction joint- the green color is from the epoxy coating.


Longitudinal construction joint
Transverse contraction joint
Tie bars
Dower bars

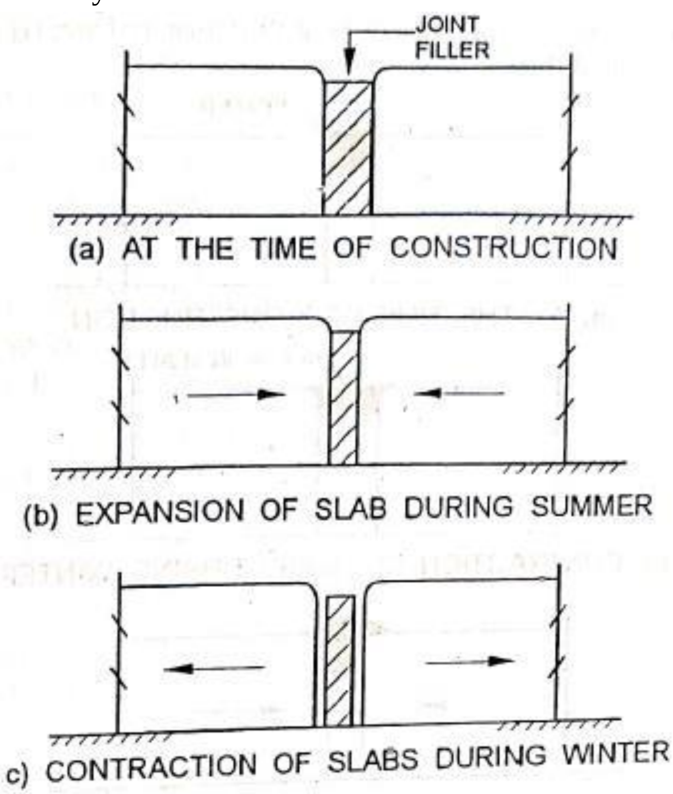

Longitudinal joints
Transverse joints

(b) Joint filler

- During hot weather when the CC pavement slabs expand, the gap provided at the expansion joints decreases and the joint filler material gets compressed; therefore the joint filler material should be ***compressible***.
- During cold season when the slabs contract, the gap opens up again and the filler material should regain its original thickness; therefore the filler material should be elastic.
- Further as the CC pavement is expected to have a long service life, it is desirable that the filler material is also durable.

Hence, the desirable properties of joint filler material are:

(viii) compressibility,
(ix) elasticity and
(x) Durability.

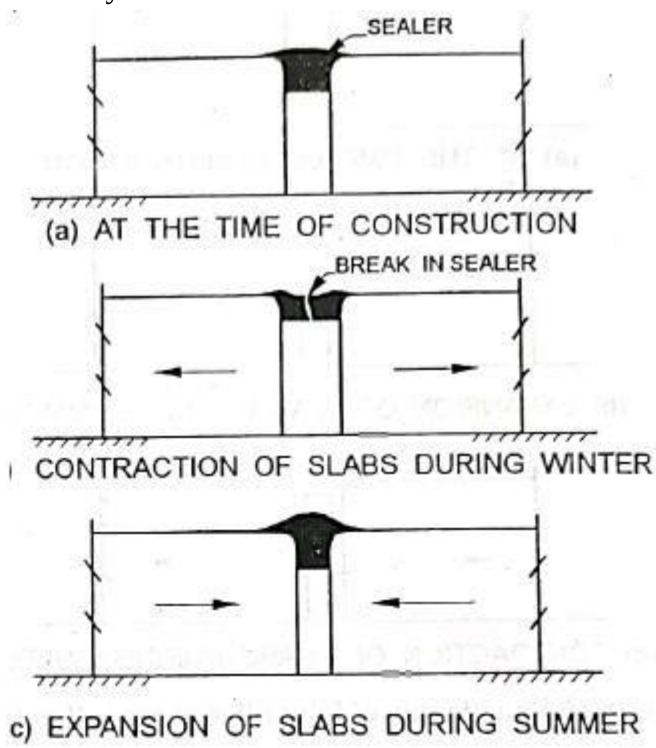


Functioning of joint filler at expansion joint

(c) Joint sealer:

- The top portion of the gap at the expansion joint above the joint filler board is sealed using a good sealant material.
- The sealer ***prevents entry of water and grit into the pavement through the expansion joint***.

- The sealing compound used should be ***impermeable and be flexible*** to accommodate the slab movements; the sealant should not flow in hot season or become brittle in cold season.
- Different types of sealing compounds are in use. ***Rubberized bitumen*** is commonly used.
- For effective sealing of joint for a long period, it is essential that the sealing compounds possess the following properties:
    - ✓ Adhesion to cement concrete edges
    - ✓ Extensibility without fracture
    - ✓ Resistance to ingress of grit
    - ✓ Durability



Functioning of joint sealer at expansion joint

Construction joints:

- The construction joint should preferably be merged with the nearest expansion joint. However if the CC pavement construction has to be suspended due to any unforeseen reason after laying a short stretch beyond the previous expansion joint, a separate construction joint may be provided along with the set of load transfer dowel bars as in the expansion joints.
- The construction method is identical to that of expansion joint given above; but it is not essential to provide the same gap as in expansion joint.

Warping joint: These are provided to relieve stresses included due to the warping. These are known as hinged joints. Longitudinal joints with tie bars fall in this class of joints. These joints are rarely needed if the suitably designed expansion and contraction joints are provided to prevent cracking.

Longitudinal joints:

Longitudinal joints are generally provided at transverse spacing of 3.5 or 3.75 in, coinciding with the traffic lane marking. Longitudinal joints are also formed as ***dummy joints similar to the contraction joints.***

The longitudinal joints are formed by cutting narrow longitudinal grooves up to the specified depth (of one fourth to one third the slab thickness) on the top of the CC slab along the identified longitudinal lines.

Steel tie bars of specified diameter and length are inserted at the mid depth of the slab at the specified intervals along the longitudinal joints.

The tie bars have two functions namely,

(viii) ***To hold the two parts of the slabs*** on either side of the longitudinal joint together and to prevent widening of the fine crack developed below the groove and

(ix) ***To act as hinge to relieve the warping stress*** developed on account of the temperature differential between the top and bottom of the CC pavement slab during the day and night.

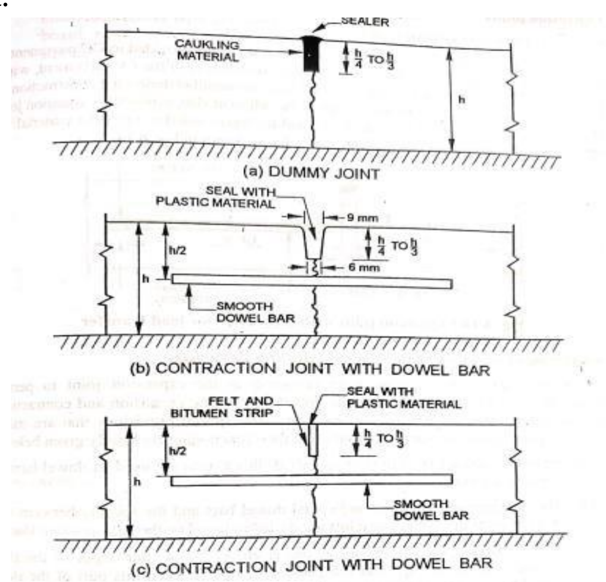
The tie bars in longitudinal joints shall consist of deformed mild steel bars. The cutting of the longitudinal groove is done by diamond cutting machines and is carried out during the period 6 to 14 hours after laying of the concrete slab.

If The CC pavement is laid using slip form paver, the tie bars are inserted automatically by the inserting the plate of the paver. If paving is done by fixed form method, the tie bars are placed at appropriate locations using the cradles or chairs prior to concreting; care is to be taken not to disturb the tie bars during paving and compaction of concrete. Thus the. Construction procedure of longitudinal joints with tie bars is identical to the construction of contraction joints with dowel bars.

Contraction joints:

- Contraction joints are ***dummy joints*** formed by cutting grooves of specified depth (one fourth to one third slab thicknesses) on the top of the slab and fine shrinkage cracks are developed in the lower portion at these locations during the initial curing period of the concrete.

- Generally the contraction joints are spaced at intervals of ***4.5 m*** or even lesser spacing as decided during the design of the CC pavement in reinforced slabs of thickness 20 cm is 14m.
- Transverse grooves are cut at the locations of contraction joints up to the specified depth using diamond saw machine cutters. Initially only narrow grooves are cut, which may be widened later as required.
- It is desirable to carryout and complete the groove cutting work of the contraction joints during the period of 4 to 16 hours, after laying the CC pavement slabs.
- The period within which the groove cutting work is to be carried out and completed depends on factors such as: (i) the temperature of CC mix at the time of laying and (ii) the ambient temperature and the relative humidity during the initial curing period up to about 24 hours after preparation of the concrete mix.
- In summer when the ambient temperature is more than 30°C, the initial cutting of the contraction joints is carried out during 4 to 8 hours after laying and in colder weather 8 to 12 hours after laying.
- Contraction joints may be constructed either as ***'plain joints' without dowel bars*** or ***with dowel bars***. Fig. shows a typical contraction joint with the groove cut on the top of the slab and the fine cracks have developed below at this location.
- There will be substantial ***load transfer across the contraction joints due to interlocking effect, even with the fine cracks***.
- Fig. illustrates the contraction joints with reinforcing dowel bars placed at the mid-depth of the CC pavement slab, are useful to minimize widening of the fine cracks at these joints due to variations in weather and movement of heavy traffic loads during the service life of the pavement.



Some types of contraction joints

Maintenance of Bituminous Surfaces

Mainly the maintenance works of bituminous surfacing consists of

ii) Patch repairs
iii) Surface treatments
iv) Resurfacing

1) **Patch repairs:**

Patch repairs are carried out on damaged or improper road surface. Localized depressions and potholes may be found in the surface layer due to defects in materials and construction. Inadequate or defective binding materials causes removal of aggregates during monsoons patching may be done on affected localized areas or sections using a cold premix.

2) **Potholes and Repairs:**

Potholes are cut to rectangular shape and are affected materials in the section is removed until the sound materials for encountered. The excavated patches are cleaned and painted with bituminous binder. A premix material is then placed in the sections. Generally cutback or the emission is used as binder. Bituminous emulsions could be used even when the pavement surface and aggregates of wet during the monsoons.

The material so placed in the potholes is well compacted by ramming to avoid any raveling the materials in the potholes are placed in the layers of thickness 6 cm or so.
It is however necessary to replace the best course material with similar new material if the failure has been detected in the base course there also definition level of the patches is kept slightly above the original level to allow for subsequent compaction under traffic.


3) **Surface Treatment:**

Excess of bitumen in the surface materials bleeds and the pavement becomes patchy and slippery, Corrugations or rutting or shoving develop in such pavement surfaces. It is customary to spread blotting materials such as aggregate chips of maximum size about 10 mm or coarse sand during summer. Necessary rolling is done to develop permanent bond between the existing surface and the new materials, after heating the surface if necessary.

The binders in the Black top surfaces also get oxidized due to aging. This develops minute cracking in the pavement surface. Bituminous wearing surface may also get worn out showing up mosaic of aggregates or rough surface to traffic and heavy rainfall. Such pavement surfaces are applied with a renewal coat such as surface dressing or seal coat. If the surface has been seriously damaged due to oxidation or volatilization of binder materials, it may be necessary to apply more than one layer of surface treatments.

4) **Resurfacing:**

In the event when the payment surface is totally worn out and develops a poor riding surface, it may be more economical to provide an additional surface course on the existing surface. In case the pavement is of inadequate thickness due to increase in traffic loads and strengthening is necessary then an **overlay** of adequate thickness should be designed and constructed.

Maintenance of Cement Concrete Roads:

It may be stated here that very little maintenance such as maintenance of joints only is needed for cement concrete roads, if they are well designed and constructed. Main defect in this type of road is formation of cracks. It is therefore necessary to examine the cracks and causes are ascertained before any remedial measure is adopted.

Treatment of Cracks:

The cracks developed in cement concrete (CC) may be classified into two groups;

(i) Temperature cracks: Which are initially fine cracks or hair-cracks formed across the slab, in between a pair of transverse or longitudinal joints, dividing the slab length into two or more approximately equal parts due to the temperature stresses like the shrinkage stress, warping stress, etc. in the slab.

(ii) Structural cracks: These are formed near the edge and corner regions of the slabs, due to combined wheel load and warping stresses in the slab.

The presence of fine cracks only as such are not harmful and do not call for immediate maintenance. As the cracks due to the shrinkage in the CC pavement start from the bottom of the slab, by the time fine cracks are visible on the top of the slab, the cracks at the bottom portion would have got widened. Due to repeated application of heavy wheel loads and the variations in temperature and moisture conditions, the cracks get widened and further deterioration becomes rapid. Once the surface water starts getting into the pavement and the subgrade through the widened cracks, progressive failure or the pavement is imminent. Therefore before these cracks get wide enough to permit infiltration of water, they should be sealed off to prevent rapid deteriorations.

The dirt, sand and other loose particles at the cracks arc thoroughly cleaned using sharp tool, still brush and pressure blower. Kerosene oil is applied on the cleaned cracks to facilitate proper bonding of the sealing material. The cracks arc then filled by a suitable grade bituminous sealing compound, heated to liquid consistency. The sealer is placed up to about 3 mm above the level of the slab along the cracks and a layer of sand is spread over it to protect the sealer temporarily.

The formation of structural cracks in CC slabs should be viewed seriously and needs immediate attention, as these indicate possible beginning of pavement failure. First the cause of the failure should be investigated. If the failure is confined to one or a few slabs only at a particular location, and in general there are no structural cracks in other slabs, the failure may be localized one due to some weak spot in the subgrade or due to localized settlement of embankment or underground drainage problem.

Maintenance of Joints: Joints are the weakest parts in CC pavements. The efficiency of the pavement is determined by the proper functioning of the joints. Majority of the failure in the CC pavements are observes at or near the joints. Therefore, utmost care is to be taken to see that the filler and sealer materials are intact at the joints. During summer the joint sealer material is squeezed out of the expansion joints due to the expansion of the stabs. Subsequently as the slabs contract during winter, the joint gap opens out and cracks are formed in the old sealer material. Therefore, periodic maintenance of the joint sealer is essential both at expansion and contraction joints as a part of routine maintenance work of the CC pavement. The opened-up joints are cleaned with brush and refilled with suitable joint sealer material before the start of the rains.

The joint filler material at the expansion joints may get damaged or deteriorated after several years of pavement life. The repair consists of removal of the sealer and deteriorated filler and sealer materials from the expansion joints cleaning up, replacement with new filler board (provided with suitable grooves cut on the bottom half at the positions of the dowel bars) and sealing the top of the joints with suitable sealer material. It will be convenient to insert the new filler board at the expansion joints during winter season when the joint opening is widest.