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.

1.1FUNCTIONS & 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.

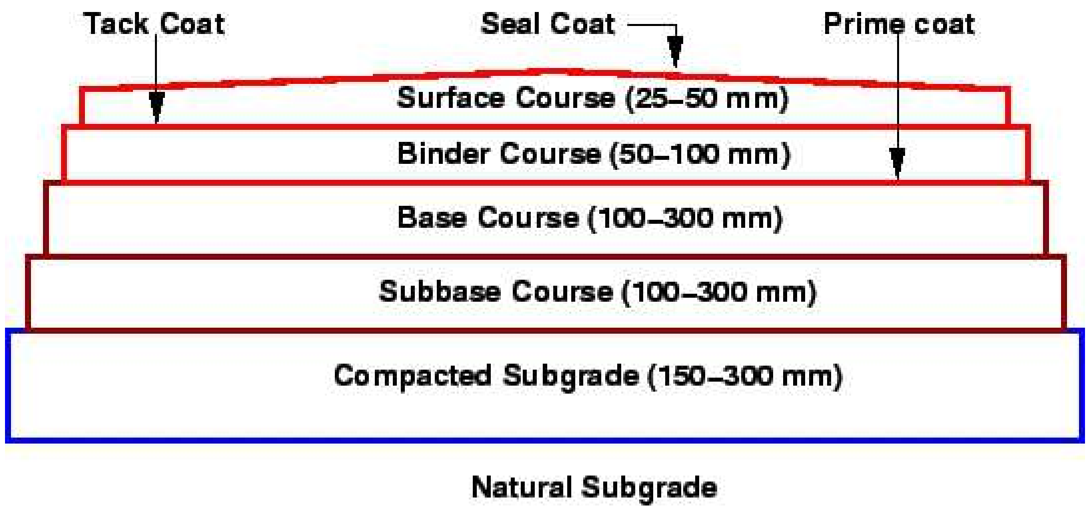


Fig 2.1.1 Cross section of Pavement.

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 pavementshave 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 PAVEMENTSAND 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 subgradeof 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 PAVEMENTSLAB:

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 WAYCOMPONENTS:

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

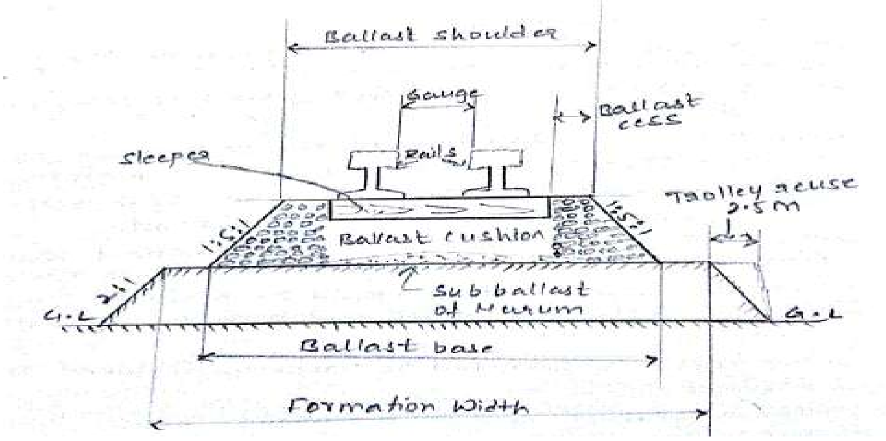


Fig 4.1 cross section of permanent way.

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.

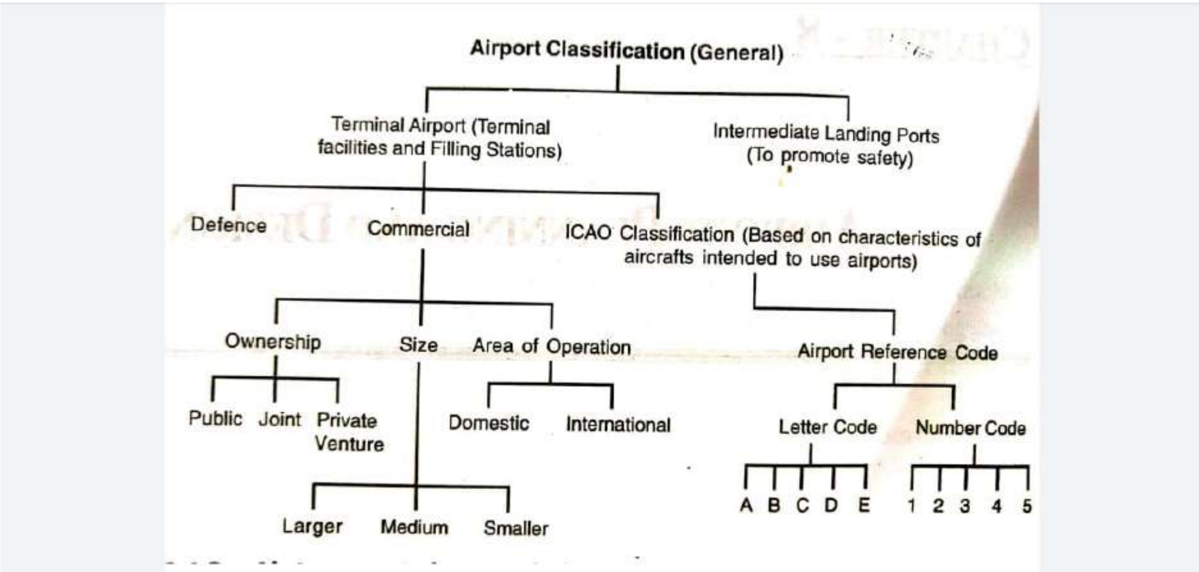


Fig 5.1 classification of airports

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.

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:

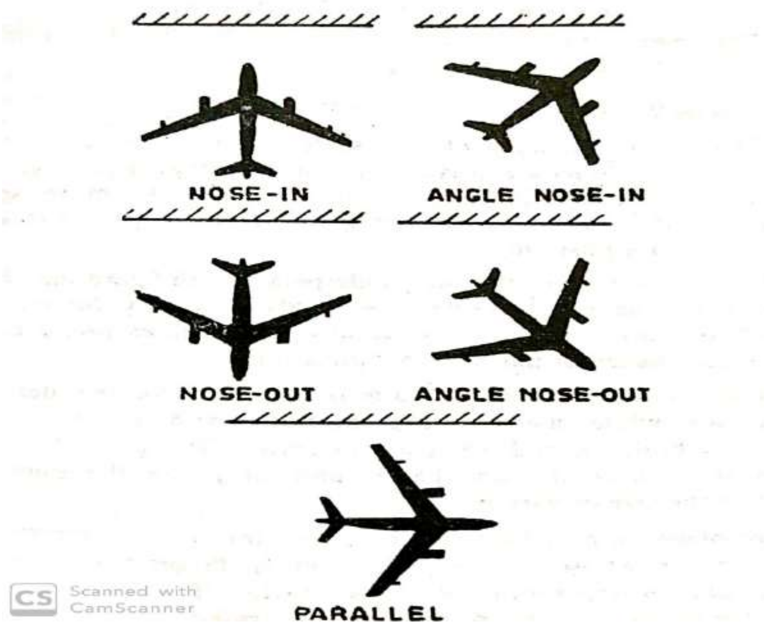


Fig 8.1.1 Parking position of aircrafts.

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 terminalbuilding.

(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 directedtowards the terminal building.

(C)PARALLEL SYSTEM

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

Thus, it is evident that no single parking configuration can beconsidered as an ideal one.A minimum clearance of 7.5 m is suggested as the desirableclearance 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 ina gate position. This time in also known as the ramp time

and it varies from few minutes for small aircraft to morethan 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 movementsper hour.In the above formula, it is assumed that eachaircraft occupying agate position, represents two aircraftmovements,landing and take off.But this may notalways be so.The aircrafts are very often brought to thegate position from a service hangar. For the design, thegate occupancy time for big aircrafts may be assumed as60 minutes.For small aircrafts requiring no.servicing, itmay be assumed as 10 minutes.

9.TERMINAL BUILDING:

Terminal building usually refers to a building mainly, usedto a buildingfor passengers, airline and administration facilities.Its layout is suchas to offer the enplaning passengers, the convenient anddirect access from the vehicle platform or street side of thestreet side of thebuildingthrough the booking and waiting rooms, to thewaitingandrooms, to the aircraft loading positions on the apron.Deplaningpassengersare also provided with a direct route from the aircraft tothebaggage claim counter and then to the vehicle platform.The operational category includes control tower, weatherbureau and other government services related to the aviation.In many cases the terminal building fulfils the function of theoperational building as well.

The various facilities provided inthe 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 buildingsfor a commercial airport.Centralization and DecentralizationIn the centralized plan, all passengers, baggage and cargo arefunnelled through a central building and are then dispersed tothe 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 thedeparting plane. The choice of a particular type of plan isgoverned by the space needed for parking of the aircrafts.When the aircraft parking area is located at an overall walkingdistance exceeding 180 m, a change from the centralizedsystem becomes necessary. Further, when the number of gatepositions (loading area required for each aircraft) required forthe individual airliner at one airport exceeds the decentralizedplan also becomes operationally uneconomical. At thissituation, another shift towards the centralization of eachindividual airline operation becomes essential. This of resultsin a series of centralized airline spaces, arranged in adecentralized pattern.

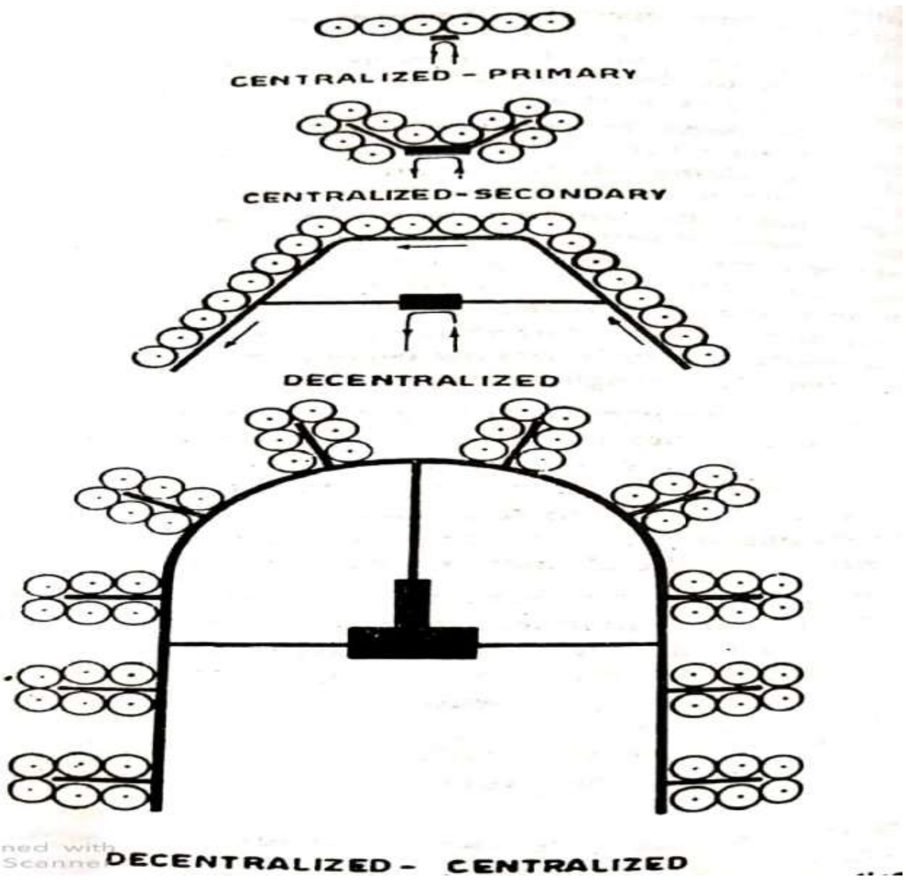


Fig 9.1.1 Centralized & decentralized systems.

10.HANGAR:

The primary function of a hangar is to provide an enclosurefor servicing, overhauling and doing repairs of the aircrafts.They are usually constructed of steel frames and covered withgalvanised iron sheets. They are also provided with machineshops and stores for spare parts. The sizeof hangar dependsupon the size of aircraft and its turning radius. Adequatelighting inside the hangar is of prime importance. Sometimeceilings of hangar and some portions of its side walls areglazed, which work as light reflectors. Construction ahangar to store large number of aircrafts may be undesirable both from economy and other considerations viz., difficulty inthe manoeuvring of aircrafts, noise nuisance, fumes, fire hazards etc. The number of hangars depends upon the peak hoursvolume of aircrafts and demand of hangars on rental basis bydifferent airline agencies. The requirements of suitablehangar site are as follows

(i)The site should be such that there is a convenient roadaccess to it from the site to the aprons and terminalbuildings.

(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 frequentstorms as this is likely to damage the hangar doors etc.

(v) Sufficient area to provide car parking facilities forworking 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

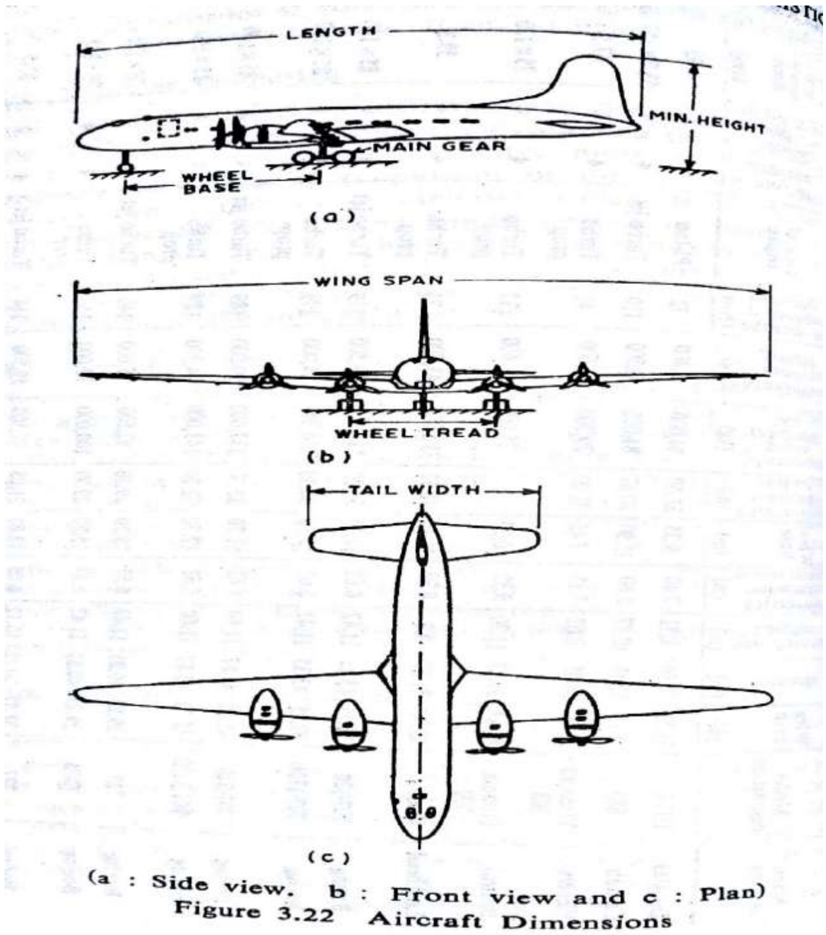
8 Jet blast

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.

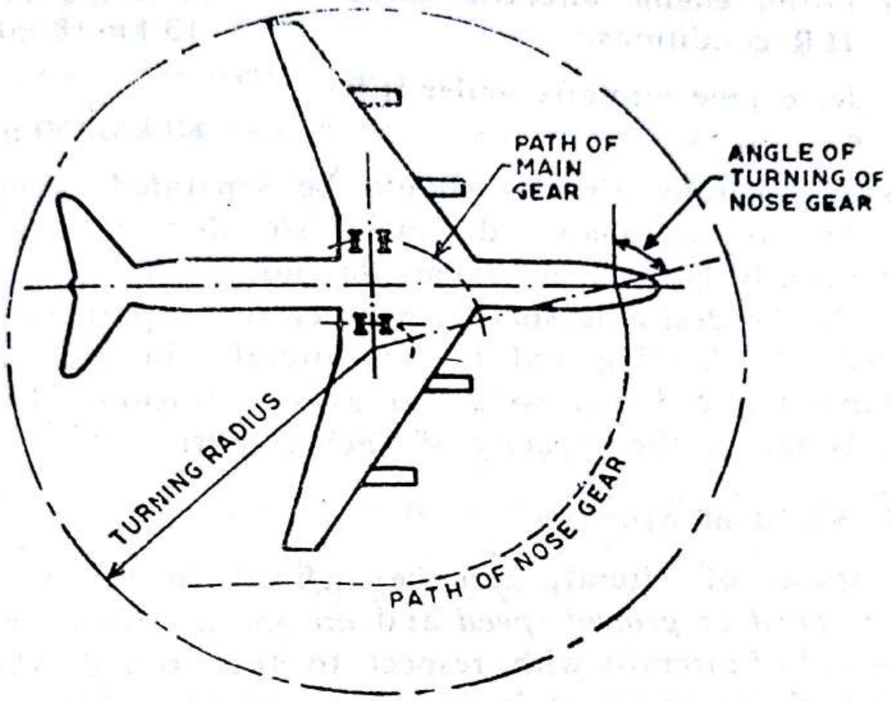


Figure 3.23 Turning Radius of Aircraft

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 landfrom the sea and from sea to the land. It is a point of change fromland carriage to sea carriage. Ships bring passengers and goods fromoverseas and discharge them in the port for convergence to inlanddestinations. 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 WATERPORT: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 havenor road-stead of navigable waters well protected naturallyor artificially from action of wind and waves, and is situatedalong sea-shore or river estuary or lake or canal connectedto sea.Draft is vertical linear immersion of ship below water surfacefor the ship to float in stable condition safely. Safe floatingrequires a standard vertical clearance between bottom partof ship and sea bed. Harbours situated on mouth of river are knowas river harbours, while those situated along lake are known as lake harbours, those situated along canals are known acanal harbours. River harbours have problems of continuous deposition which if not overcome by maintenance dredgingnavigability, 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 neededharbours are broadly classified as:

(i) Natural harbours.

(ii) Semi-natural harbours

(iii) Artificial harbours

(i)NATURAL HARBOURS:

Naturalformations affording safe discharge facilities for ships onsea coasts, in the form of creeks and basins, are callednatural harbours.In other words, natural harbour is an inlet protectedfrom storms and waves by natural configuration of land.With the rapid development of navies engaged eitherin commerce or war, improved accommodation and facilitiesfor repairs, storage of cargo and connected amenities hadto 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 ofpopulation, development of the area, etc. have made thenatural harbours big and attractive. Bombay and Kandlaare examples of natural harbours.

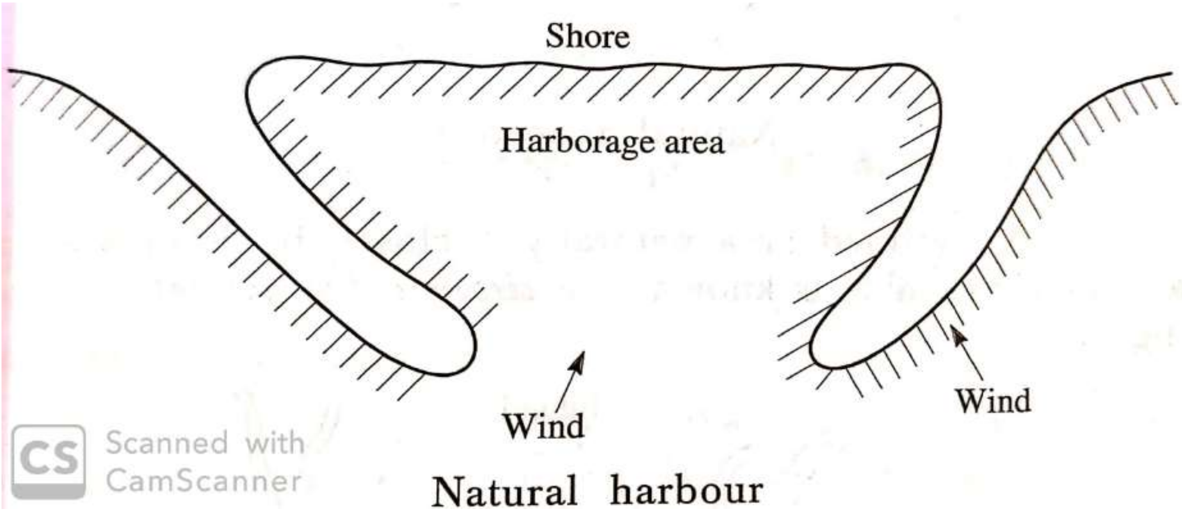


Fig 13.1 Natural harbour

(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.

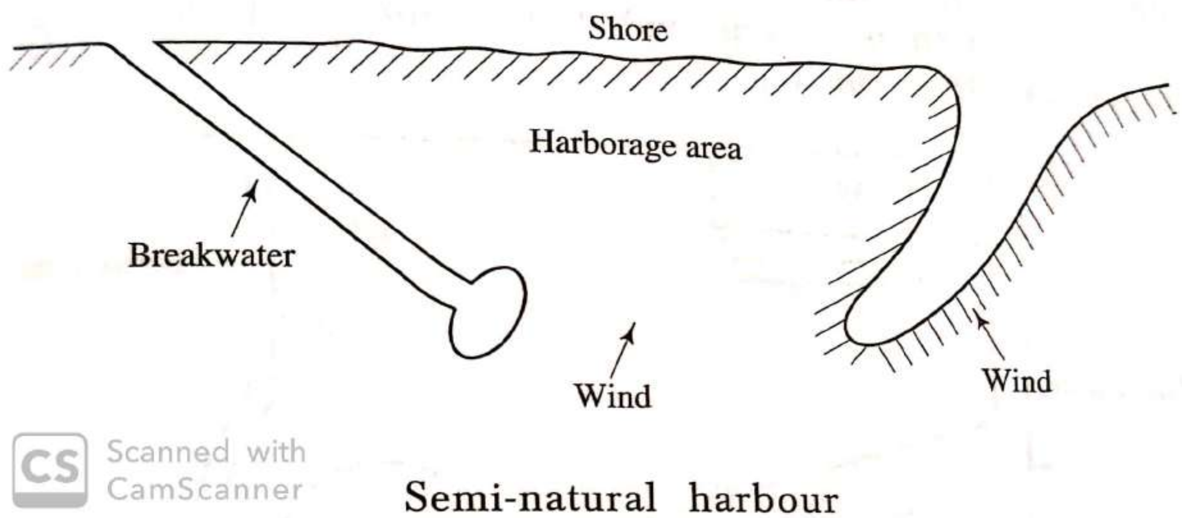
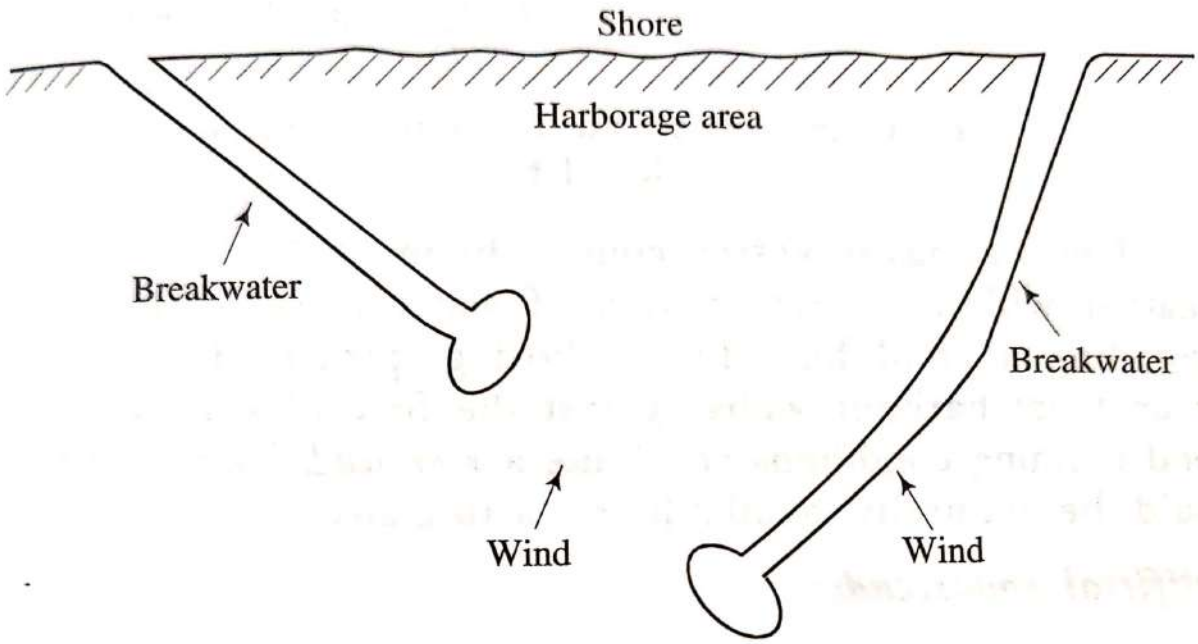


Fig.13.2 semi-natural harbour

(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.



Artificial harbour

Fig 13.3 Artificial harbour

(2)CLASSIFICATION DEPENDING UPON THE UTILITY:

From their utility, harbours are furtherinto 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 shipsunder stress of weather conditions will need quick shelterand immediate repairs. All types of naval craftsmall and big will need such refuge in emergency and hence refuge harbours shouldprovide commodious accommodation. Modern big ships willrequire 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 unloadingcould be done with advantage in calmer waters.

Commercial harbours could be situated on coasts orestuaries of big rivers or even on inland river coasts. Theydo not normally have any emergency demand like a harbourof refuge and practically the size and number of ships usingsuch 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 despatchfacilities for the perishable fish catch like railwaysidings and roads.

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

(iv) MILITARYHARBOURS:

Requirements of military harbour:

These harbours are the naval bases which are meantto accommodate the naval vessels. They serve as supplydepots also. Bombay and Cochin harbours have naval bases.

(v)MARINA HARBOURS:

Marina is harbour providing facilities offuel, food, showers, washing machines, telephone, etc. forSmall 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 ormore berths and cater large boats. At times space available islimited and hence create a long waiting list to get berth.

(b) SMALL MARINAS:

The small marinas have less than100 berths. Marinas in general are located on fresh watersor on coastal waters.

(3) CLASSIFICATION OF HARBOUR BASED UPON THE LOCATION:

The layout of a harbour is greatly influencedby its location and based on the location, harbours arefurther 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 theshore of lake is known as lake harbour. If the lake is largethe conditions are similar to those in ocean exceptthat tidal action does not occur.

(III) RIVER OR ESTUARY HARBOUR:

The harbour constructedalong the banks of river is known as river or estuary harbour. Rivers and estuaries create the main transportationroute to join the hinterland and the sea. The best possibilitiesfor sea-going navigation are offered by the lower reachesof a river where the tides are determining the hydraulicconditions. Hence, many sea-ports have been Constructedon a tidal river.

(IV) SEA OR OCEAN HARBOUR:

The harbour located onthe 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 witheconomy and efficiencyby 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 restof the country through rail and highways so thatthe commodities can be transported to and fromthe port easily and quickly.

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

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

(vi) It should be a place of defence and for resistingthe sea-borninvasion.

(vii) It should command valuable and extensive trade.

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

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

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

(xi) The passage to open sea must have sufficientdepth and width and it should be suitably markedto aid navigation.

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

Assignment-Cum-Tutorial Questions

UNIT I

INTRODUCTION

Assignment-Cum-Tutorial Questions

I) Objective Questions

1)__________ &__________ main types of pavements.

2)_________________ is the first layer from bottom of pavement.

3)Bitumen is extracted from__________________.

4) ______________ is the top layer in Flexible pavement.

5)______________ is the top layer in Rigid pavement.

6)______________no of layers in Flexible pavement.

7)______________no of layers in Rigid pavement.

8)_________ to _______ mm size aggregates are used in subgrade layer.

9)One of the following part belongs Permanent way? [ ]

a) Runway b) Apron c) Sleepers d)Surface course

10)On which part rail can travel? [ ]

a) Rails b) Gauge c) Sleeper d) Ballast

11) Distance between two rails is said to be [ ]

a) Rails b) Gauge c) Sleeper d) Ballast

12) currently using sleepers are made up of [ ]

a) Wood b) Iron c) Concrete d) Teak

13) The main function of Runway is [ ]

a) Take off b) Landing c) both a & b d) none.

14) The main function of Taxiway is___________________

15) Loading & Unloading of Passengers and cargo takes place on ___________ area.

16) The main function of Service Hanger is_____________________

17)_________________ & _______________________ systems are used in Terminal Building.

18) Booking of tickets, Waiting halls are located in ___________________ of airport.

19)Units of Noise is________________

20) __________ is a place where Cargo is stored.

21)In Warm water port, water _______________

22)Harbours are classified into ______________types.

23) Based on Utility, Harbours are classified into ___________ types.

24)Based on location, Harbours are classified into ___________ types.

25)Marine Harbours are classified into _________ types.

26)____________ no of berths in Large marine Harbours.

27)____________no of berths in Small marine Harbours.

28)Flexural strength is ______________ Rigid pavements.

29) Maintenance cost is __________ in Flexible pavements.

30)Load is distributed _________ to _________ in Flexible pavements.

31)_______________________________________ test are adopted for evalulation pf subgrade.

Descriptive Questions:

1)Classify the Types of Pavements with their functions and requirements?

2) Describe the components of Flexible pavements?

3)Describe the components of Rigid pavements?

4)Write the difference between Flexible and Rigid pavements?

5)Define permanent way and its components with neat sketch?

6) Demonstrate Runway and its elements?

7)Write the difference between Runway and Taxiway?

9)What is Apron? Evaluate the factors that decides the size of apron?

10) Define Terminal building? What are facilities provided in Terminal building?

11) Discuss Centralized and Decentralized systems in Terminal Building with neat sketch?

12)Discuss about the Service Hanger in Airport?

13) Discuss the Aircraft characteristics?

14) Define Harbor. Explain the classification of Harbors?

15)Define Port. Demonstrate the classification of Ports?

16)Investigate the Requirements of Good ports?

UNIT II

HIGHWAYDEVELOPMENT 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 HIGHWAYPLANNING

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

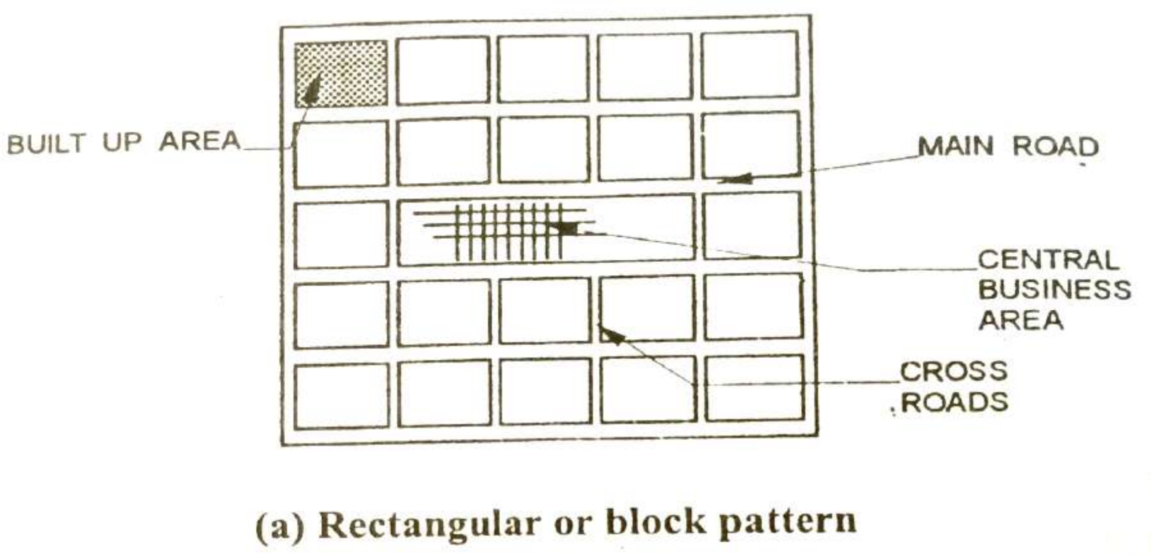
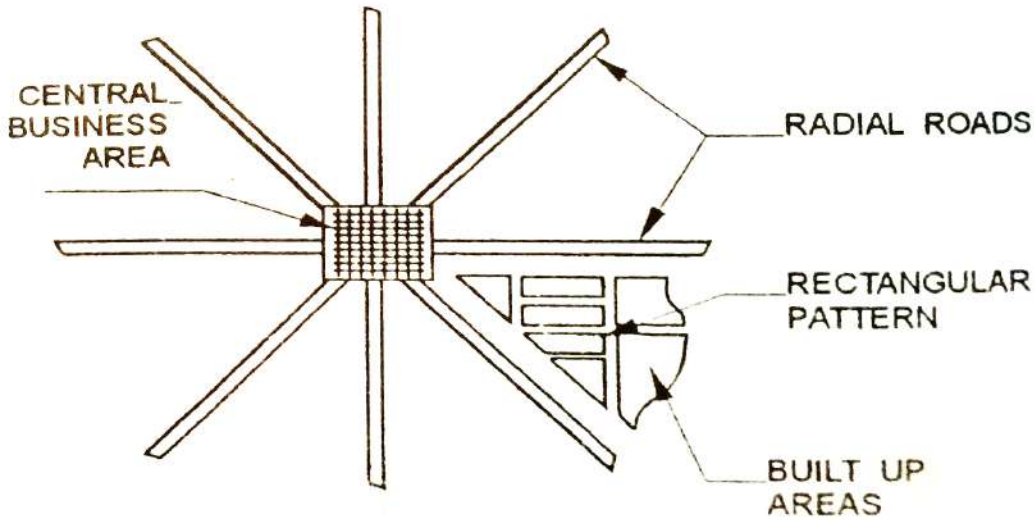
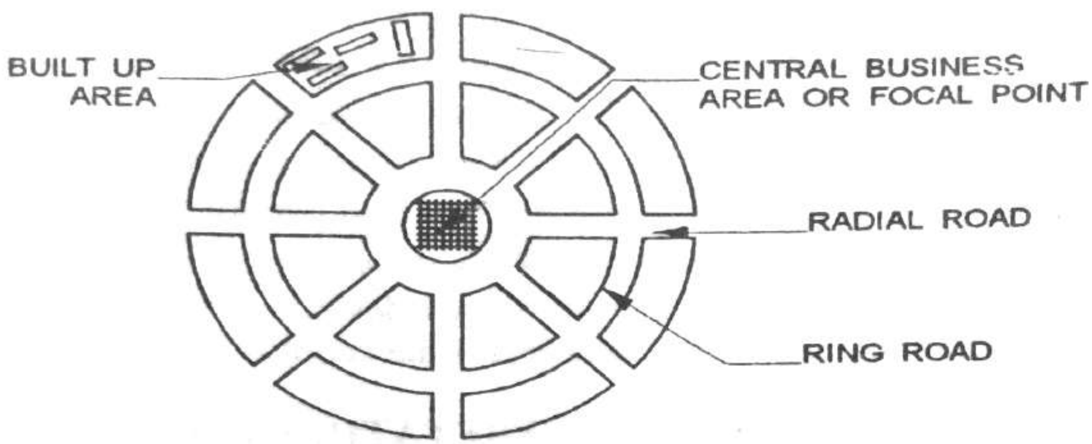


Fig. 4.1(a) Rectangular or block pattern



(c) Radial or star and block pattern

Fig 4.1(b) Radial or star and block pattern



(d) Radial or star and circular pattern

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

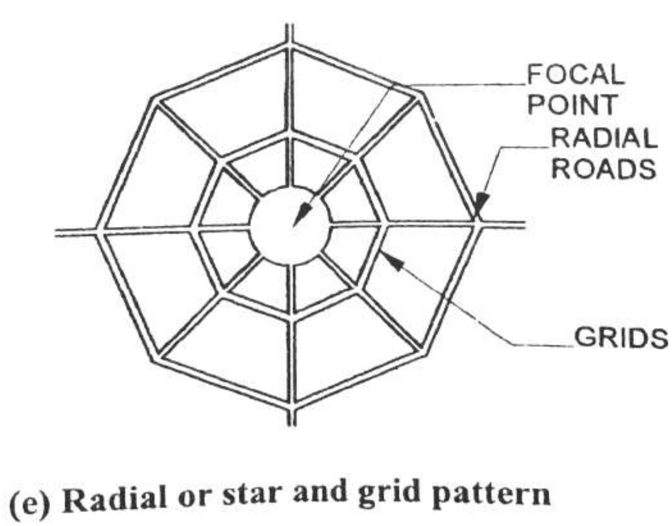


Fig. 4.1 (d) Radial or star and grid pattern

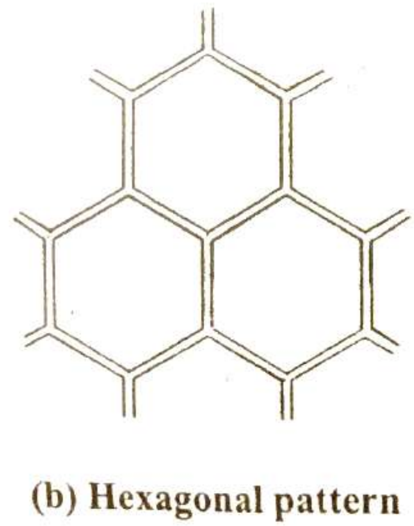


Fig. 4.1 (e) Hexagonal pattern

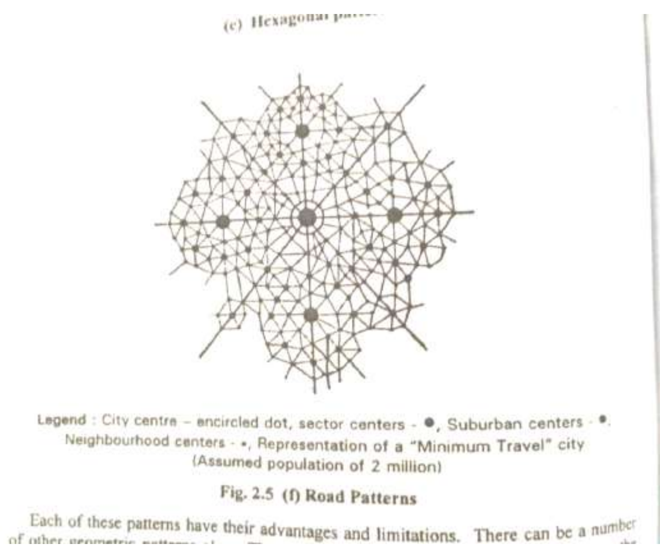


Fig. 1.1 (f) Minimum travel Pattern

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.

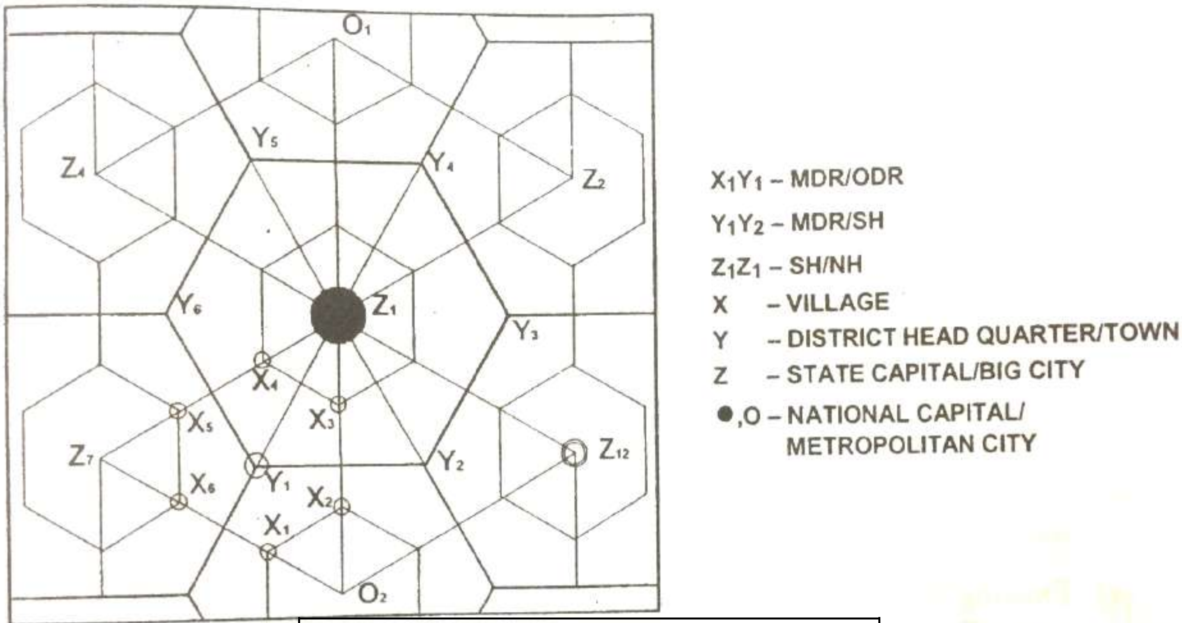


Fig. 1.1(g) concept of star and Grid pattern

In the star and grid pattern Fig. 1.2 points X are assumed to be villages, points Y the towns and points Z represent the district headquarters or state capitals. 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

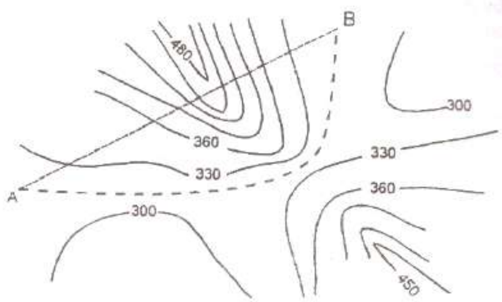
v. Other considerations

6.2.1 Obligatory Points

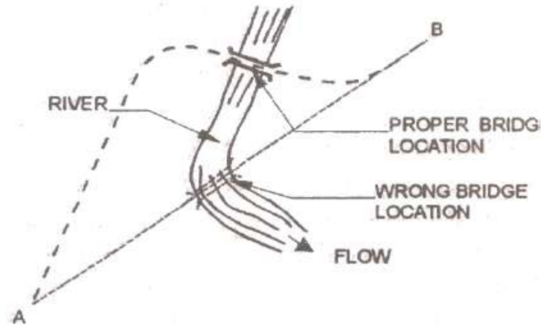
These are control points governing the alignment of the highways. These control points may be divided broadly into two categories.

(i) Points through which the alignment is to pass

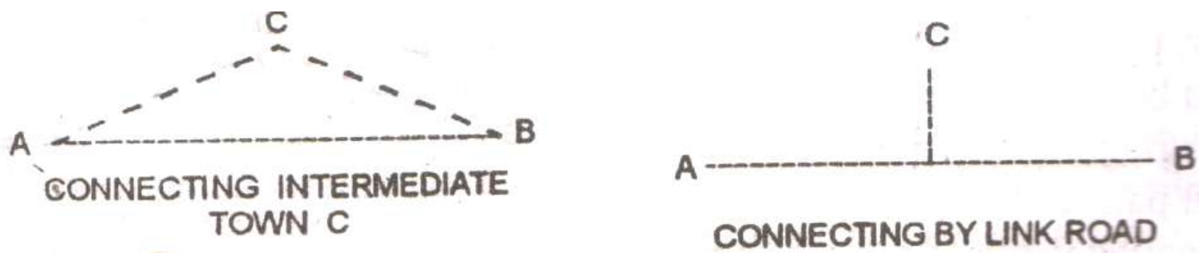
(ii) Points through which the alignment should not pass



(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.

Assignment-Cum-Tutorial Questions

A. Questions testing the remembering / understanding level of students

I) Objective Questions

1) The period for which 1st 20 year road plan adopted is_______________________

2) Second twenty year road plan is also called as ____________________________

3) Third twenty year road plan is ended in which year ___________________

4) What is meant by Highway alignment?

5) Classify the roads according to first twenty year plan.

6) Mention the different types of road network patterns?

7) What is the need of map survey?

8) What is meant by reconnaissance survey?

II) Descriptive Questions

1) Briefly outline highway development in India.
2) What are the significant recommendations of Jayakar committee?
3) Explain the salient features of the first twenty year plan
4) Explain the significant features of the second twenty year plan.
5) Explain the significant features of the third twenty year plan
6) What are the various requirements of the ideal alignment? Discuss them briefly
7) Explain obligatory points. With sketches, discuss how these control the alignment
8) Explain different types road patterns and their suitability
9) What are the different approaches of road classification?

B. Question testing the ability of students in applying the concepts.

I) Multiple Choice Questions

1) CRF stands for

a) Central Research federation b) Central Road Federation

c) Central Road Fund d) Central Resource Fund

2) On which year IRC came into existence?

a. 1930 b. 1934 c. 1939 d. 1948

3) The road network pattern which is less convenient for traffic operation is

a. Rectangular pattern b. Hexagonal Pattern

c. Radial and Star c. Radial and Circular

4) What is the target density road length for the Nagapur Road plan?

a. 16 Km/100 Km^2 b. 32 Km/100 Km^2

c. 48 Km/100 Km^2 d. 46 Km/100 Km^2

7) What is the target density road length for the Bombay Road plan?

a. 16 Km/100 Km^2 b. 32 Km/100 Km^2

c. 48 Km/100 Km^2 d. 46 Km/100 Km^2

8) What is the target density road length for the Lucknow Road plan?

a. 16 Km/100 Km^2 b. 32 Km/100 Km^2

c. 48 Km/100 Km^2 d. 46 Km/100 Km^2

9) Nagpur road congress classified the roads based on ____________

a) Location b) function c) construction material d) traffic volume

10) The depth up to which soil sampling is to be done is________________ below the ground level in detailed survey

a. 2m b.3m c.4 m d. 5 m

II) Descriptive Questions

1) Classify the roads based on Lucknow road development plan?

2) What are the different outcomes of Jayakar committee recommendations?

3) What is meant by State Highway?

4) What is the Necessity of Highway planning?

5) What are different types and objectives of conducting preliminary survey and when we use Rapid approach?

6) Explain about radial pattern in roads and need of ring roads

7) Briefly explain the engineering surveys needed for locating a new highway

8) Outlines the different factors controlling the alignment.

9) Explain how the final location and detailed survey for highway are carried out.

D. Question testing the ability of students in competitive exams.

1. For transportation purposes in India, the first preference is given to

   a. Airlines   b. Roads   c. Shipping   d. Railways

2. The star and grid pattern of road network was adopted in _______________ Road plan

3. I.R.T.D.A (Indian Roads and transport Development Association) was set up in Bombay in _______________

3. The Central Road Research Institute is controlled by ___________ Ministry

   a. Shipping and Transport   b. Science and Technology

   c. Planning   d. Finance

4. The Motor Vehicle Act was enacted in

   a. 1930   b. 1934   c. 1939   d. 1948

5. A road connecting two towns is called.

   a. Country road   b. urban road   c. Highway   d. None of these

TRANSPORTATION

ENGINEERING

UNIT-III

Objective
:

To acquire design principles of Highway geometrics and pavements

Syllabus: HIGHWAYGEOMETRIC 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)

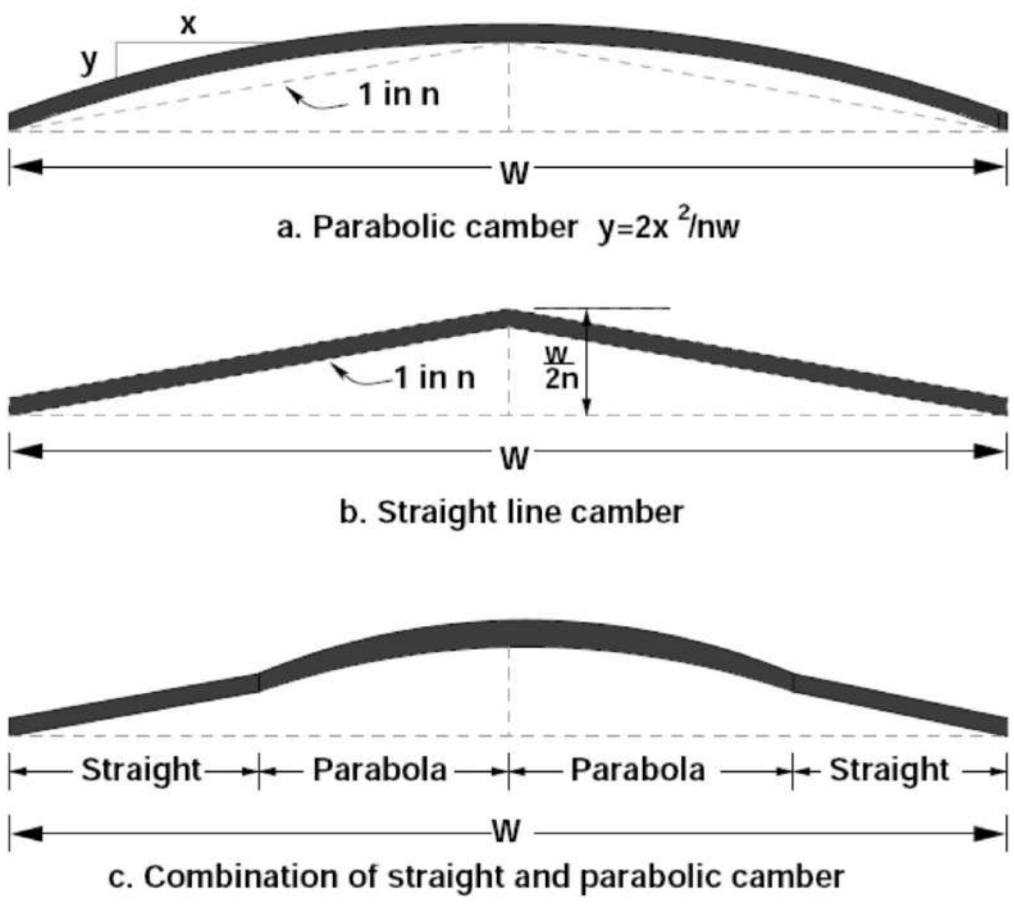


Fig. 2.1 Different types of camber

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.

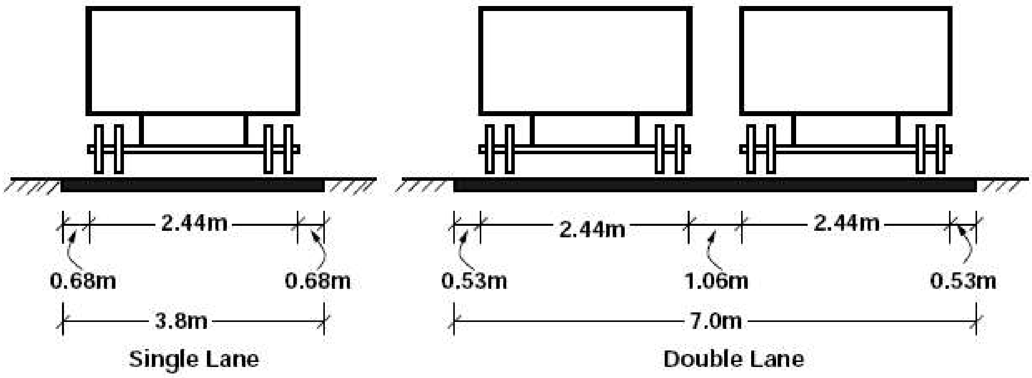


Fig. 2.2 Lane width for single and two lane roads

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.

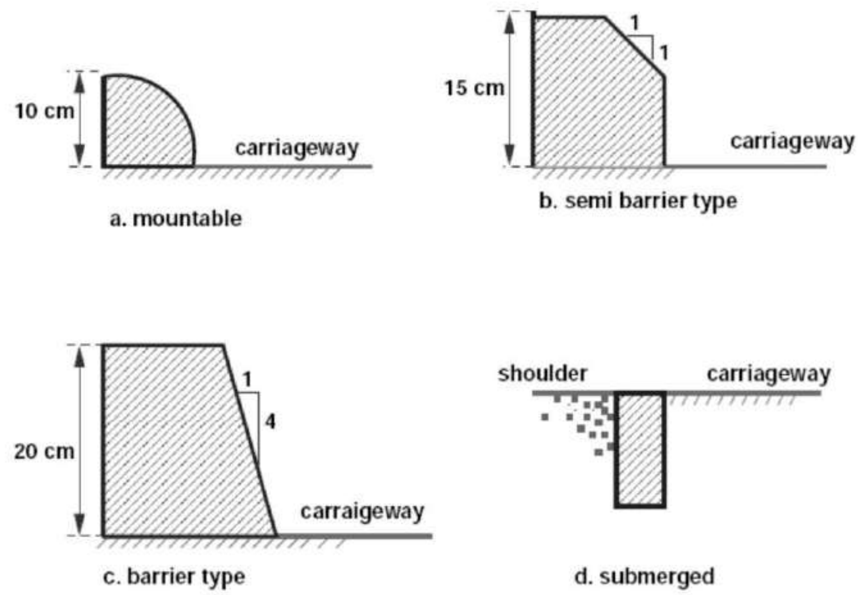


Fig. 2.3 Different types of Kerbs.

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

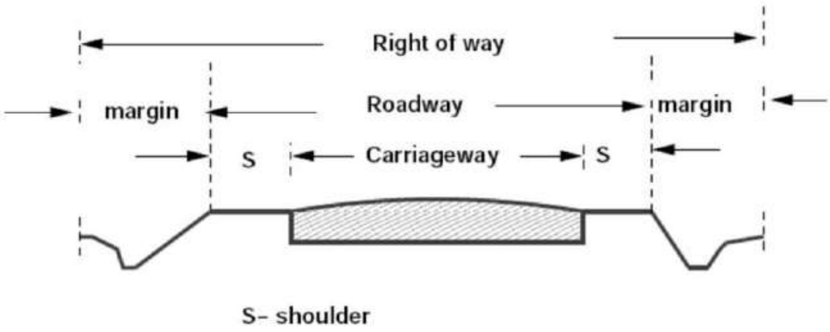


Fig. 2.4 A typical Right of way (ROW)

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

1 ？？？？ 1 ？？？？

2 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:

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

[illegible]2[illegible]

[illegible] = [illegible] 2[illegible] ([illegible] + 0.01[illegible])

Similarly the braking distance can be derived for a descending gradient. Therefore the general equation is given by Equation [illegible] 2 [illegible]

[illegible] = [illegible] + [illegible] 2 [illegible] ([illegible] ± 0.01 [illegible])

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

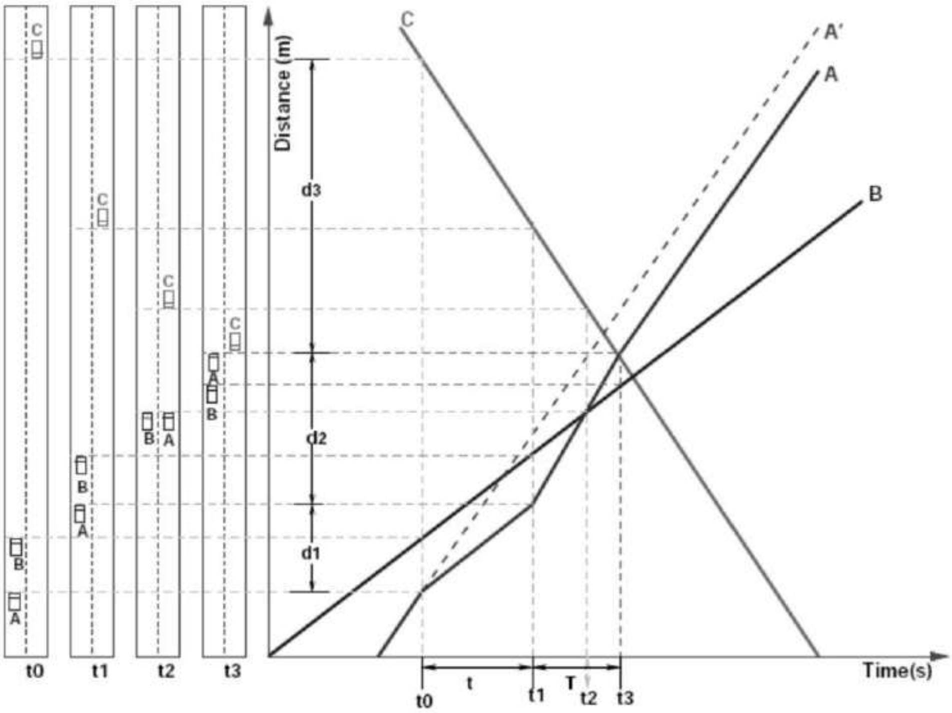


Fig. 2.5 Time-space diagram: Illustration of overtaking sight distance

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:

$$d_2 = v_bT+$$

$$T=$$

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)

$$= d_1+d_2+d_3$$

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).

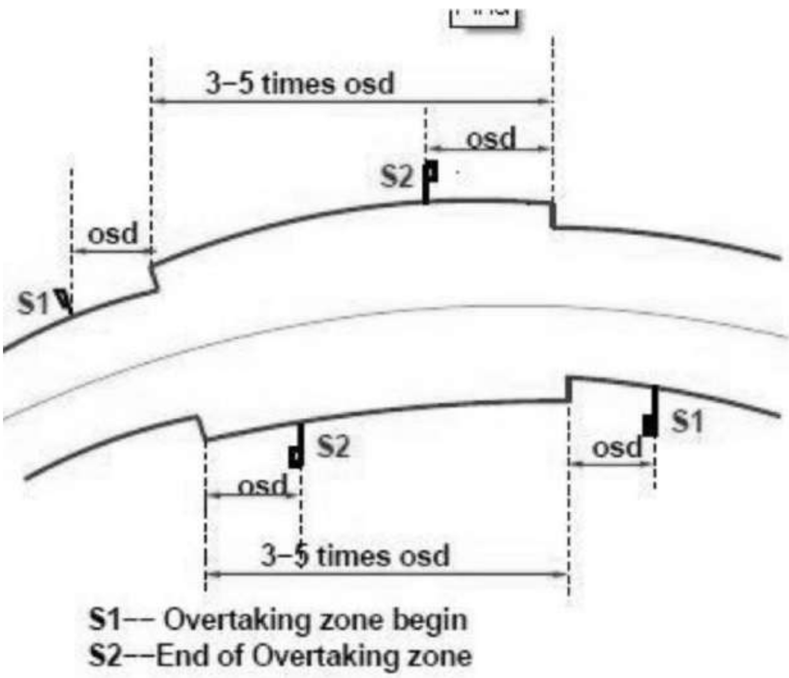


Fig. 2.6 Overtaking zones

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

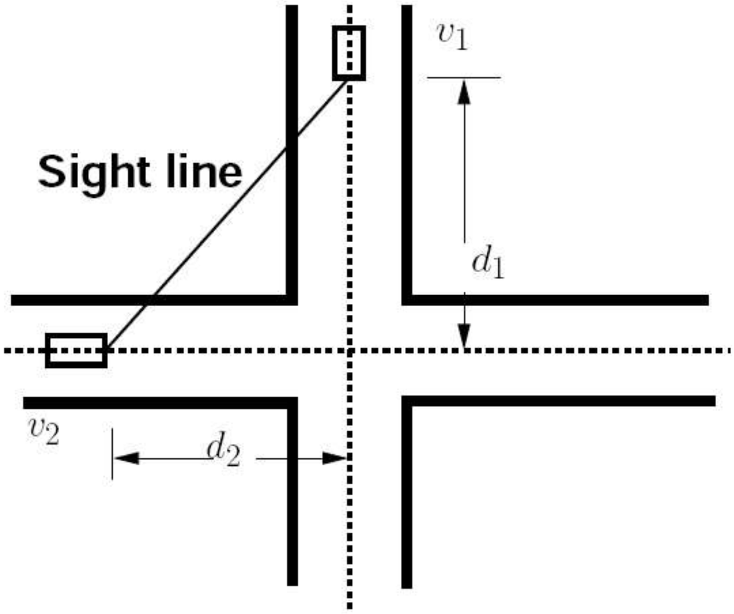


Fig. 2.7 Sight distance at intersections

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.

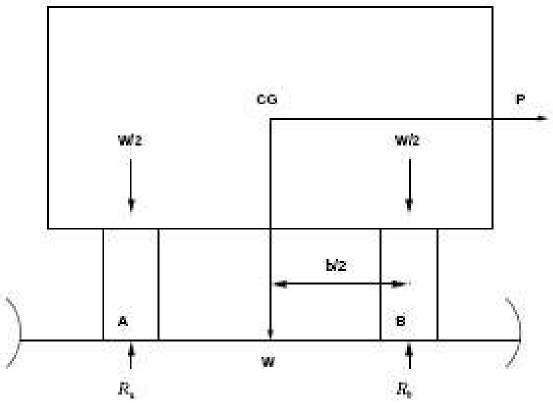


Fig. 2.8 Effect of horizontal curve

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/m2 is given by

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,

$$\square\square h = \square\square \frac{\square\square}{2} \square\square \frac{\square\square}{\square\square} \square\square = \frac{\square\square}{2h}$$

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

$$\frac{??}{??} = ?? = \frac{?2?}{????}$$

and for safety the following condition must satisfy:

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:

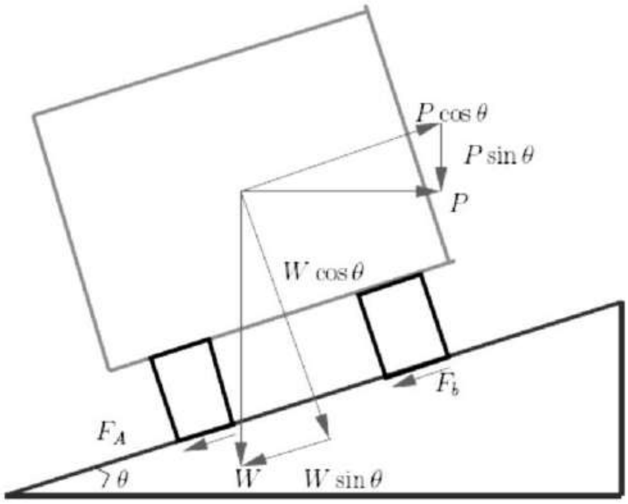


Fig. 2.9 Analysis of superelevation

- 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,

[illegible] = [illegible] sin [illegible] + [illegible] + [illegible]

= [illegible] + [illegible] ([illegible] + [illegible])

= [illegible] + [illegible] ([illegible] + [illegible]) [illegible]

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:

$$\frac{\square\square}{\square\square} = \frac{\square\square\square\square + \square\square\square\square}{1 - \square\square\square\square\square\square\square\square\square}$$

We have already derived as expression for P/W.

[illegible]2[illegible] = [illegible] + [illegible]

[illegible] 1 − [illegible]

Relatively f tan[illegible] can be neglected.

[illegible]2[illegible]

[illegible]1 + [illegible]2 = [illegible]

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.

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.

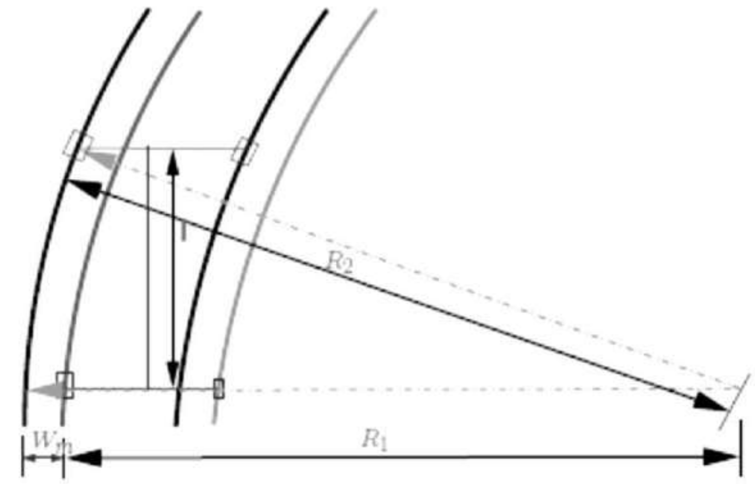


Fig. 2.10 Extra-widening at a horizontal curve

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:

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}:

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 α 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

(3)

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

$$\text{[illegible]} = \frac{80}{75+\text{[illegible]}} \quad (C_{min} = 0.5,\ C_{max} = 0.8)$$

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:

$$L_{s2} = eN(W + W_e)$$

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

$$[illegible]_{[illegible]3} = 2.7\,[illegible]^{2}_{4}\,[illegible]$$

[illegible]

□2□□ □□

+

2□□ 9.5√□□

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

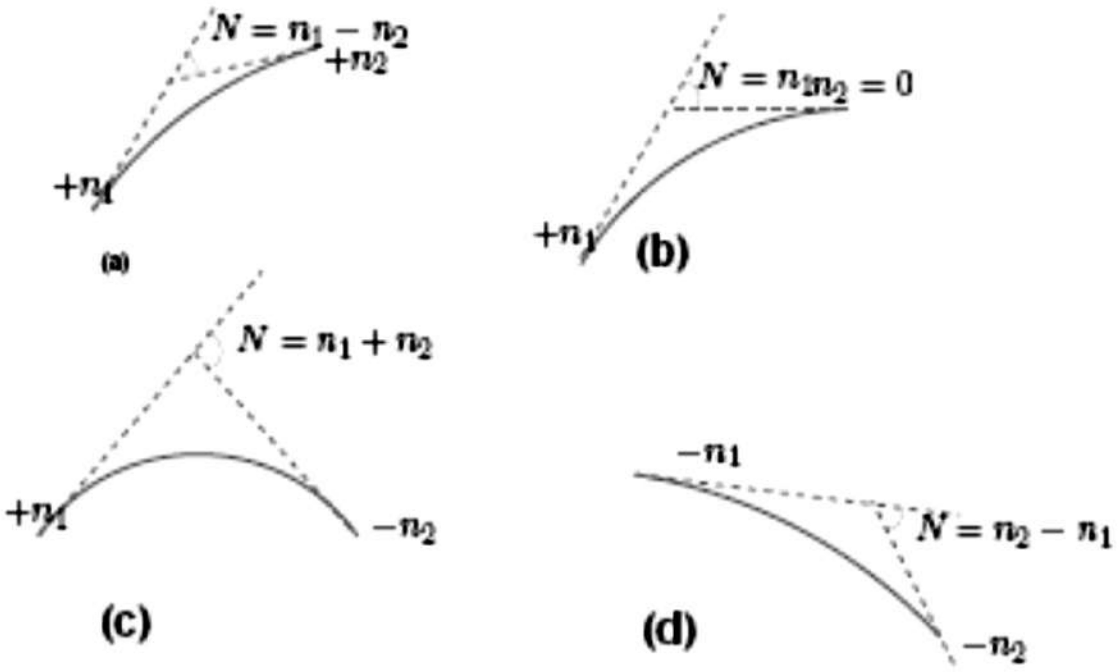


Fig. 2.11 Types of summit curves

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.

$$\square\square = 2\square\square - \frac{(\sqrt{2}h_1 + \sqrt{2}h_2)^2}{\square\square}$$

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

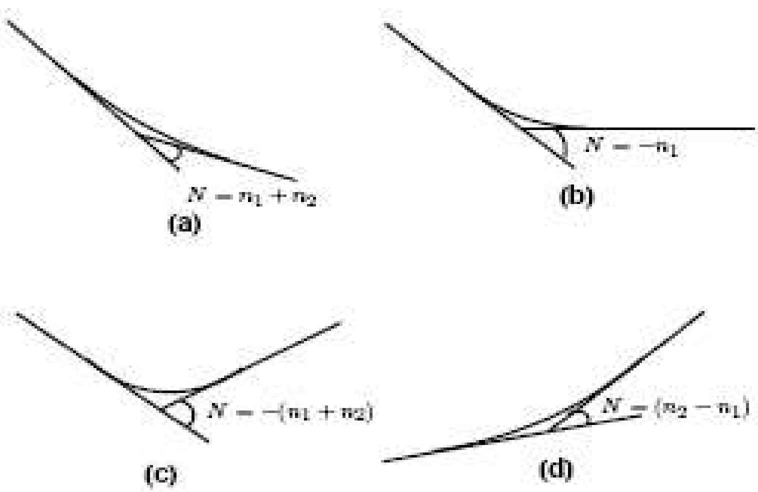


Fig. 2.12 Types of valley curve

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.

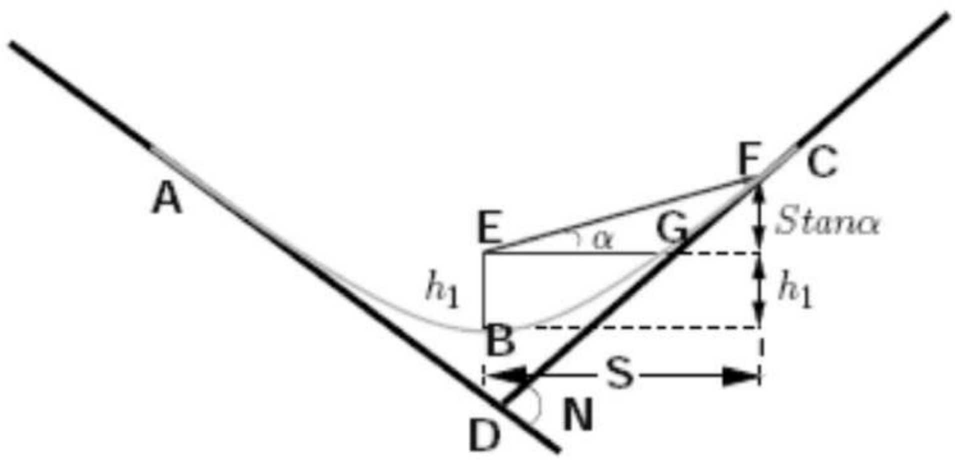


Fig. 2.13 Valley curve, case1, L > S

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.

$$[illegible] = \frac{[illegible]\,[illegible]^{2}[illegible]\,[illegible]}{2h_1 + 2[illegible]t\tan[illegible]\,[illegible]}$$

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.

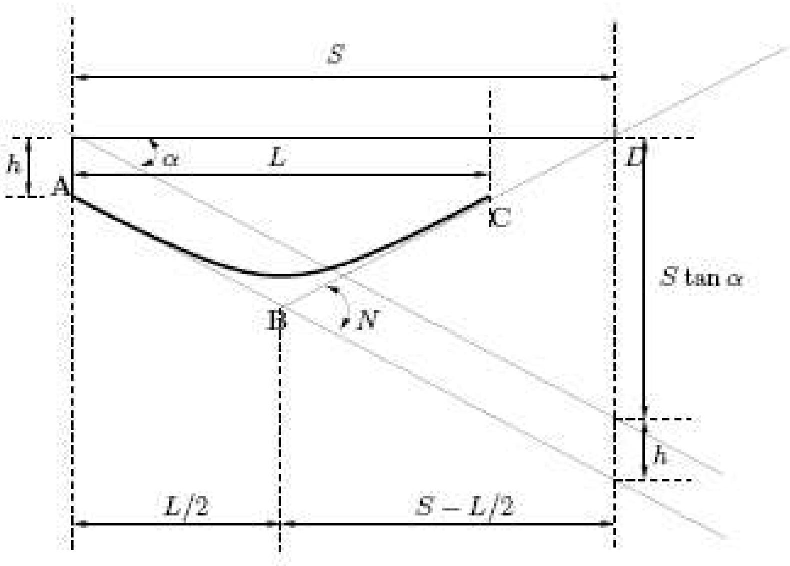


Fig. 2.14 Valley curve, case 2, S > L

Unit III HIGHWAY GEOMETRIC DESIGN

A. Questions testing the remembering / understanding level of students

I) Objective Questions

1) List the various geometric elements to be considered in highway design.
2) Illustrate the purpose of camber?
3) Define gradient.
4) What is the camber to be provided for thin bituminous pavements?
5) What is the width of single lane road?
6) What is meant by Right of Way?
7) Stopping sight distance depends on__________
8) Where de we provide overtaking zones?
9) What is meant by ruling gradient?
10) What is the minimum overtaking zone length that needs to be provided?
11) Recall the formula for finding the extra widening required on the horizontal curves.
12) Tell the formula for intermediate sight distance?
13) List out the formula for finding the transition length of the curve based on IRC recommendations?
14) Outline off tracking?
15) Recall the maximum super elevation and friction that can be provided in horizontal curves?
16) What is meant by lag distance? Is it a constant?

II) Descriptive Questions

1) Explain the role of pavement surface characteristics in highway geometric design. State the factors affecting the friction between pavement and tyres of vehicles?
2) Explain PIEV theory.
3) Develop an expression for the stopping sight distance. Explain the importance of SSD.
4) Develop an expression for the overtaking sight distance for a multilane road.
5) Explain super elevation. List out the factors on which the design of super elevation depends?
6) Outline the objectives of widening of pavement on horizontal curves? Also developan expression for mechanical widening of road.
7) Explain summit and valley curves and the various cases when these are formed while two different gradients meet.
8) Measure the safe stopping sight distance for design speed of 60 kmph for

a. Two way traffic on a two lane road.

b. Two way traffic on a single lane road.

Assume co-efficient of friction as 0.35 and assume that driver is in normal condition.

9) The speed of overtaking and overtaken vehicles are 80 and 60 kmph, respectively on a two way traffic road. If the acceleration of the overtaking vehicle is 0.99 m/s^2. Estimate the following

a. safe overtaking sight distance

b. minimum length of overtaking zone and draw a sketch of the overtaking zone and show the positions of sign posts.

10) A two lane road with design speed 80 kmph has horizontal curve of radius 540m. Design the rate of super elevation for mixed traffic. By how much should the outer edges of the pavement be raised w.r.t the centre line, if pavement is rotated w.r.t the center line and the width of the pavement at the horizontal curve is 7.5m?

11) Measure the values of ruling minimum and absolute minimum radius of horizontal curve of a NH in plain terrain. Assuming ruling design speed and minimum design speed values as 100 and 80 kmph respectively.

12) Predict the extra widening required for a pavement of width 7m on a horizontal curve of radius 250m if the longest wheel base of vehicle expected on the road is 7m. design speed is 70 kmph.

13) Propose the length of transition curve using the following data. Design speed 65 kmph, radius of circular curve 220m, allowable rate of introduction of super elevation (pavement rotated about the centre line) is 1 in 150. Pavement width including extra widening 7.5m.

14) Imagine a vertical summit curve is formed at the intersection of two gradients +3.0 and -5.0 percent. Design the length of summit curve to provide a stopping sight distance for a design speed of 80 kmph. Assume any other data required.

15) Imagine a valley curve is formed by a descending grade of 1 in 25 meeting an ascending grade of 1 in 30. Design the length of valley curve to fulfil both comfort condition and head light sight distance requirements for a design speed of 80 kmph. Assume allowable rate of change of centrifugal acceleration C = 0.6 m/s3.

B. Question testing the ability of students in competitive exams.

1. The total length of a valley formed by two gradients -3% and + 2% curve between the two tangent points to provide a rate of change of centrifugal acceleration 0.6 m/sec2, for a design speed 100 km ph, is

A. 16.0 m

B. 42.3 m

C. 84.6 m

D. none of these.

2. If R is the radius of a main curve and L is the length of the transition curve, the shift of the curve, is

A. L/24 R

B. L2/24 R

C. L3/24 R

D. L4/24 R

E. L/12 R

3. A district road with a bituminous pavement has a horizontal curve of 1000 m for a design speed of 75 km ph. The super-elevation is

A. 1 in 40

B. 1 in 50

C. 1 in 60

D. 1 in 70

E. none of these

4. The period of long term plan for the development of roads in India, known as Bombay Plan (Aug. 1958), is

A. 5 years

B. 10 years

C. 15 years

D. 20 years

E. 25 years

5. The maximum safe speed on roads, depends on the

A. type of the highway

B. type of road surface

C. type of curves

D. sight distance

E. all the above

6. What is the reaction time of ideal driver?

7. What is grade compensation? Where do we provide it?

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.

HIGHWAYMATERIALS

1.MATERIALS FOR FLEXIBLE PAVEMENTLAYERS:

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.

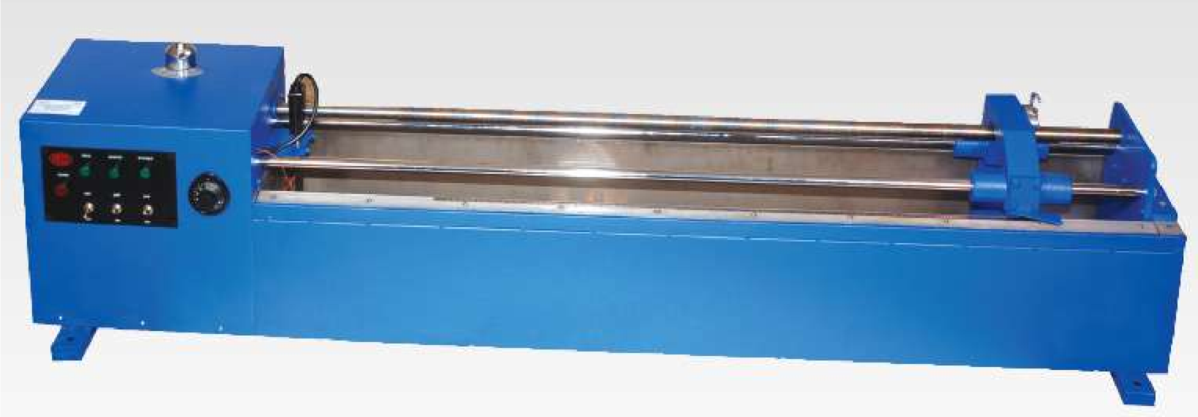


FIG.3.2.DUCTILITY TEST

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

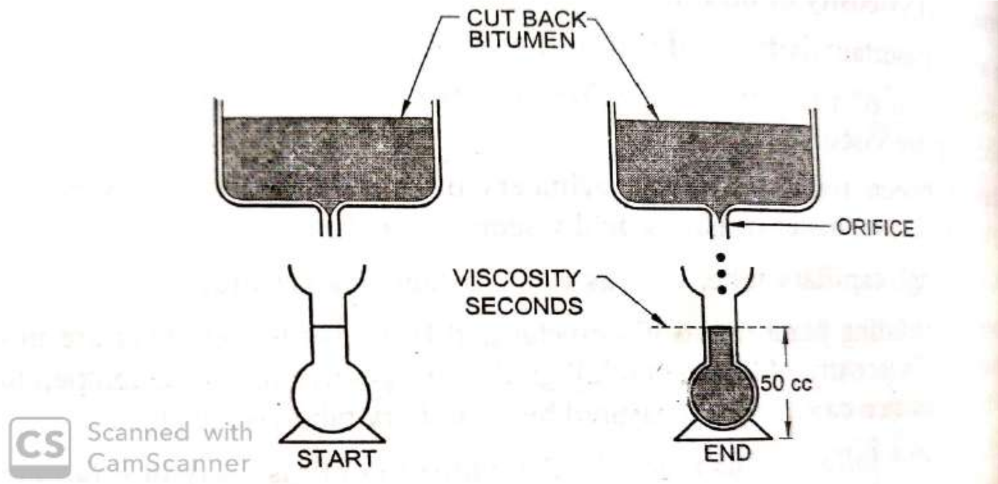


FIG.3.4.1 CONCEPT OF VISCOSITY TEST

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

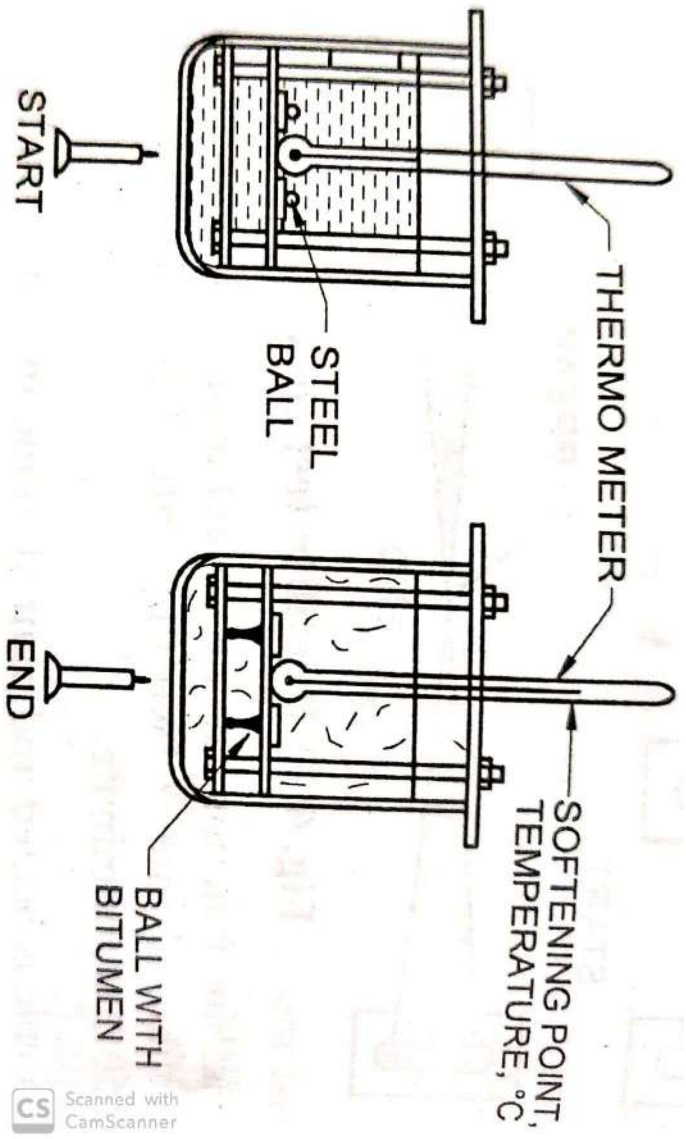


FIG.3.6.1 CONCEPT OF SOFTENING POINT

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

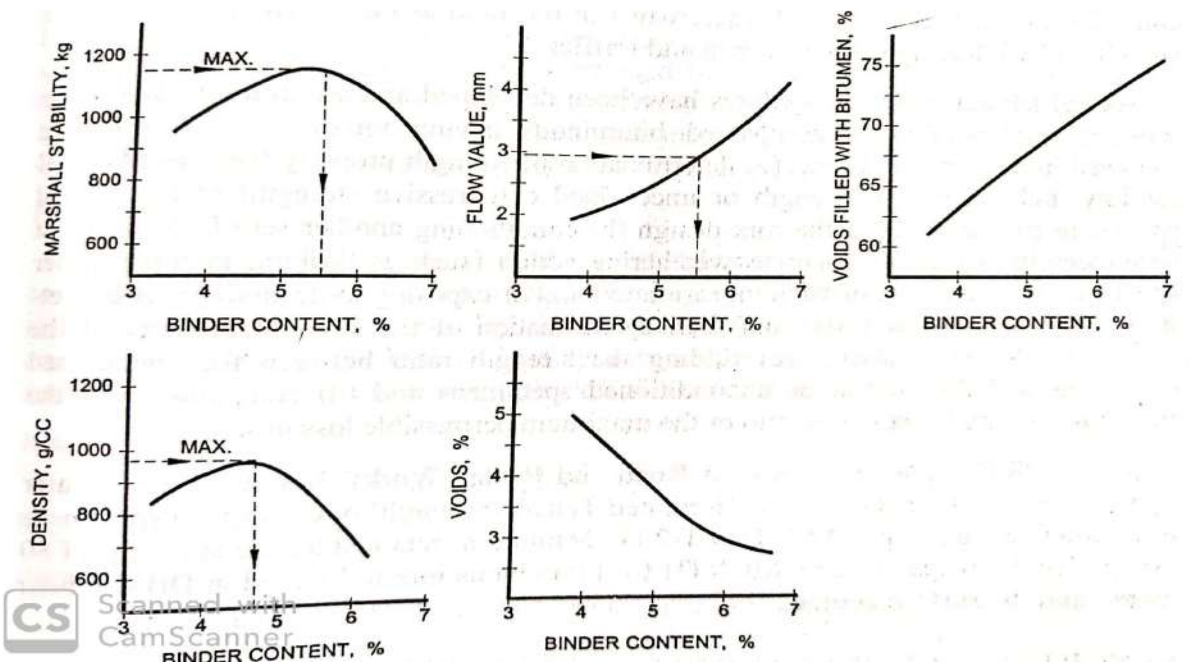


FIG.3.7.1.BITUMINOUS MIX DESIGN NY MARSHALL METHOD

Assignment-Cum-Tutorial Questions

UNIT IV

HIGHWAY MATERIALS

Assignment-Cum-Tutorial Questions

I) Objective Questions:

1) Aggregate impact test is to assess the ________________ of aggregates
2) Los Angeles abrasion test is to evaluate the__________________ of aggregates
3) Aggregate crushing test to determine _______________________ of aggregates.
4) If the aggregate impact value is 20 to 30 %, then it is classified as_______________.
5) Penetration test on bitumen is used to determine______________
6) The maximum limit of water absorption for aggregate suitable for road construction is_______%
7) The softening point of bitumen is usually determined by ______________ test
8) Full form of BIS_______________________________________
9) Full form of AASTHO__________________________________
10) Full form of MORTH___________________________________
11) The shape of aggregate particles is determined by ___________________________________
12) Formula for Aggregate crushing value is___________________________

13) Weathering property of aggregate is determined by ______________________ test.

14) __________________ no of balls are used in abrasion test.

15) ____________________no of revolutions rotated in abrasion test.

16) ____________________ tonnes of load is applied on aggregates in aggregate crushing test.

17) Marshall stability test on bituminous mix specimen is carried at standard temperature of________ C.

18) Ductility of bitumen performed at standard temperature of _________________C.

19) Full form of V.G______________________

20) At ________________ temperature most of paving bitumen grades remain in semi-solid or in plastic state.

Gate Questions

A bitumen sample has been graded as VG30 as per IS : 73-2013. The '30' in the grade means that (2018) [b]

(A) penetration of bitumen at 25C is between 20 and 40

(B) viscosity of bitumen at 60C is between 2400 and 3600 Poise

(C) ductility of bitumen at 27C is more than 30 cm

(D) elastic recovery of bitumen at 15C is more than 30%

The following observations are made while testing aggregate testing for its suitability in pavement construction 2017[a]

i.Mass of oven-dry aggregate in air = 1000g

ii. Mass of saturated surface-dry aggregate in air = 1025g

iii. Mass of saturated surface-dry aggregate under water = 625g

based on above observations the correct statement is

(A) Bulk specific gravity of aggregate = 2.5 and water absorption = 2.5%
(B) Bulk specific gravity of aggregate = 2.5 and water absorption = 2.4%
(C) Apparent specific gravity of aggregate = 2.5 and water absorption = 2.5%
(D) Apparent specific gravity of aggregate = 2.5 and water absorption = 2.4%

Maximum size of aggregate in base course is [b]
a) 25 mm
b) 50 mm
c) 40 mm
d) 30 mm

The specific gravity of bitumen lies between [c]
a) 0.8& 0.9
b) 0.95& 0.97
c) 0.97&1.02
d) 1.02&1.05

The specified method for bitumen mix in India is [b]
a) Hveem
b) Marshalls method
c) Hubbard method
d) Super paver mix method

Descriptive Questions:

(1) Explain highway materials & its properties?

(2) Judge the desirable properties and tests for aggregates?

(3) Demonstrate about Aggregate crushing test?

(4) Discus about find Toughness of aggregates?

(5) How do we determine strength of aggregates?

(6) Judge the desirable properties and tests for bitumen?

(7) List out various tests on Bitumen?

(8) Demonstrate Marshall Method of mix design?

(9) Write a report on Ductility test of bitumen?

(10) Explain about Penetration test of bitumen?

(11) Discuss about viscosity of bitumen?

(12) Describe softening point of bitumen?

(13) Investigate binder content of bitumen?

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 PAVEMENTDESIGN 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.

$$A = P(1 + r)^x \qquad (4.6)$$

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.

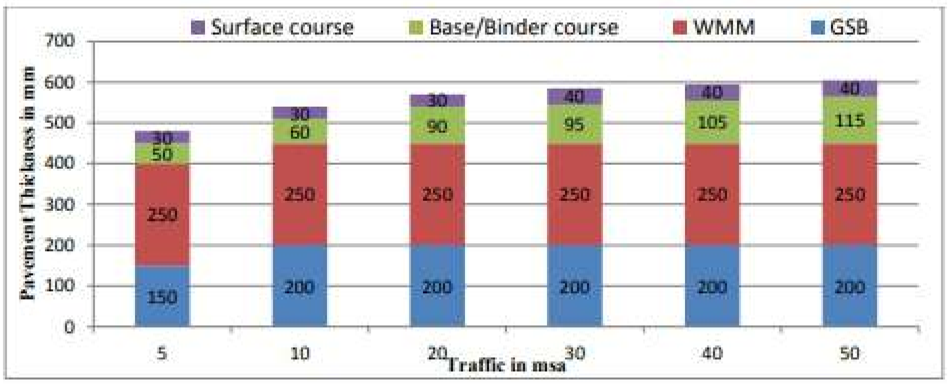


Figure 12.4 Catalogue for pavement with bituminous surface course with granular base and sub-base - Effective CBR 8% (Plate-4)

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 (3.0x10^5), 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

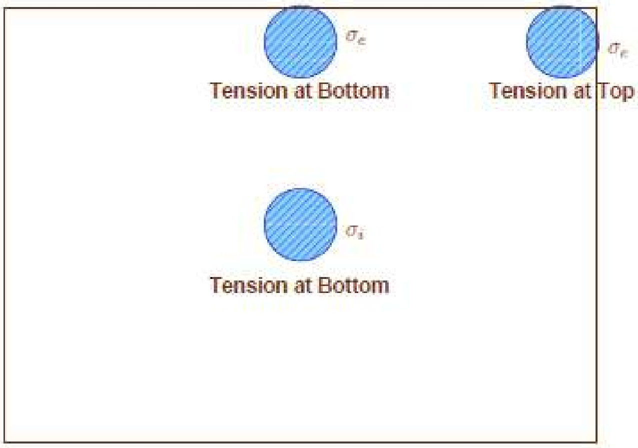


Figure : Critical stress locations

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 $\sigma_{tc}, \sigma_{te}, \sigma_{ti}$ in kg/cm^2 respectively and given by the equation

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

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

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

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 PAVEMENTFAILURES

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.

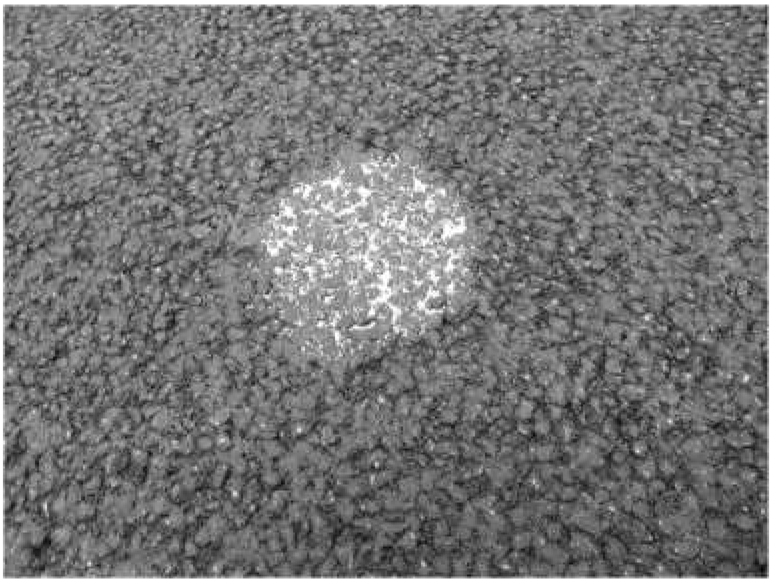
• *High severity cracks* *(> 1/2 inch wide and numerous cracks)*. Remove and replace the cracked pavement layer with an overlay.

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.

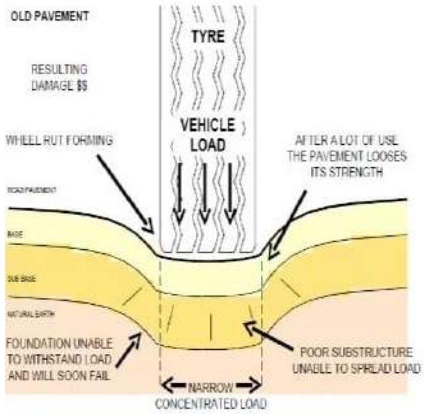
Problem: Ruts filled with water can cause vehicle hydroplaning, can be hazardous because ruts tend to pull a vehicle towards the rut path as it is steered across the rut.

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, there is no need to soak the subgrade soil before testing, as the field moisture content may never exceed the OMC at which the soil is compacted

(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

CONSTRUCTION OF RIGID PAVEMENT

List of Contents

- Rigid Pavement
- Materials
- Basic Components of Rigid Pavement
- Types of Rigid Pavement
- Methods of Construction
- Types of Joints

RIGID PAVEMENTS

- 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.

- 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.

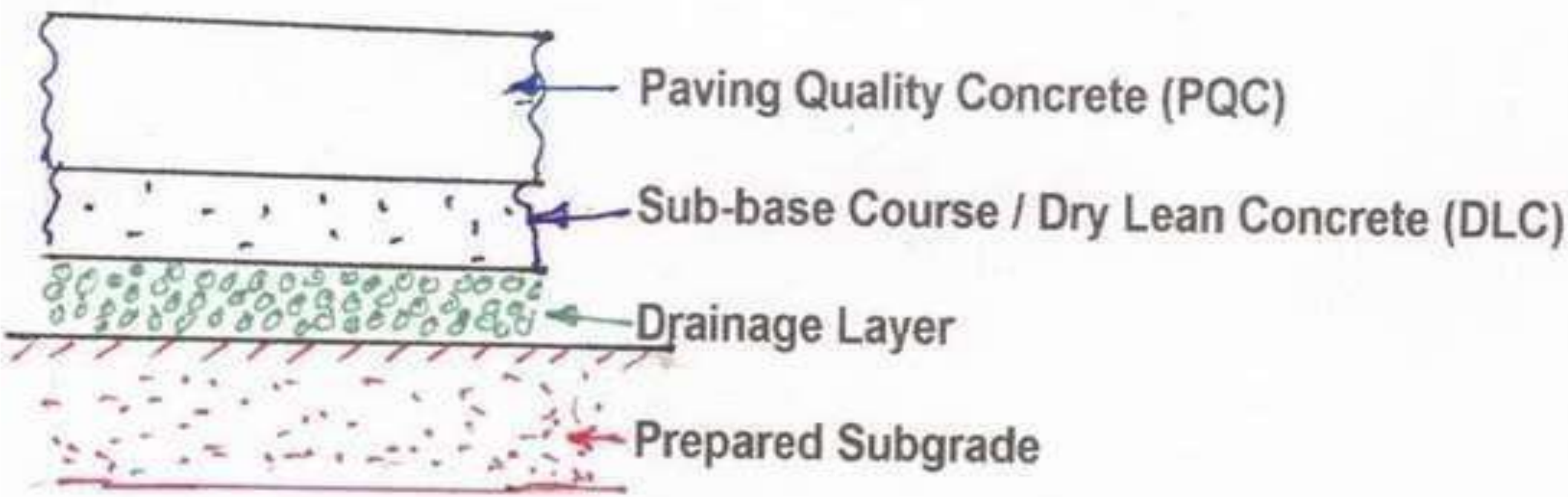
WHERE ARE RIGID PAVEMENT PROVIDED?

Rigid pavements are usually provided when road stretch is subjected to adverse conditions:

- Very heavy rainfall.
- Poor soil conditions
- Poor drainage
- Extreme climatic conditions

- Combinations of some of these conditions which may lead to development of cracks in pavements.

Components of CC pavement



Components of Cement Concrete Pavement

MATERIALS FOR CONSTRUCTION OF CC PAVEMENTS :

1.Portland cement

: OPC of grade43.

OPC of grade53.

Portland pozzolona cement with fly ash (20 per cent)

Portland slag cement.

2. Coarse aggregates :

Combined flakiness and Elongation index	<35%
Water absorption	<3%
soundness	For Na_2So_4 <12% For $MgSo_4$ <18%

Fine aggregates :

Clean natural sand / crushed stones /combination of both.

It should be free of clay ,coal and ignite. Well graded with 100% passing 10 mm sieve.

Water: Water used for mixing and curing of concrete shall be clean and free from injurious amount of oil, salt, acid, vegetable matter or other substances harmful to the finished concrete. It shall meet the requirements stipulated in IS:456.

Chemical Admixtures : conforming to IS:9103 and IS:6925 shall be permitted to improve workability of the concrete and/or extension of setting time, on satisfactory evidence that they **will not have of any adverse effect** on the properties of concrete with respect to strength, volume change, durability and have no deleterious effect on steel bars.

Reinforcement

- Steel dowel bars (rounded) with yield strength 240 MPa are used for the **load transfer** across in expansion and construction joints.

- Plain or twisted steel bars are used as tie bars(prevent opening up of longitudinal bars) are used as tie bars at longitudinal joints.

Basic Components of CC Pavements:

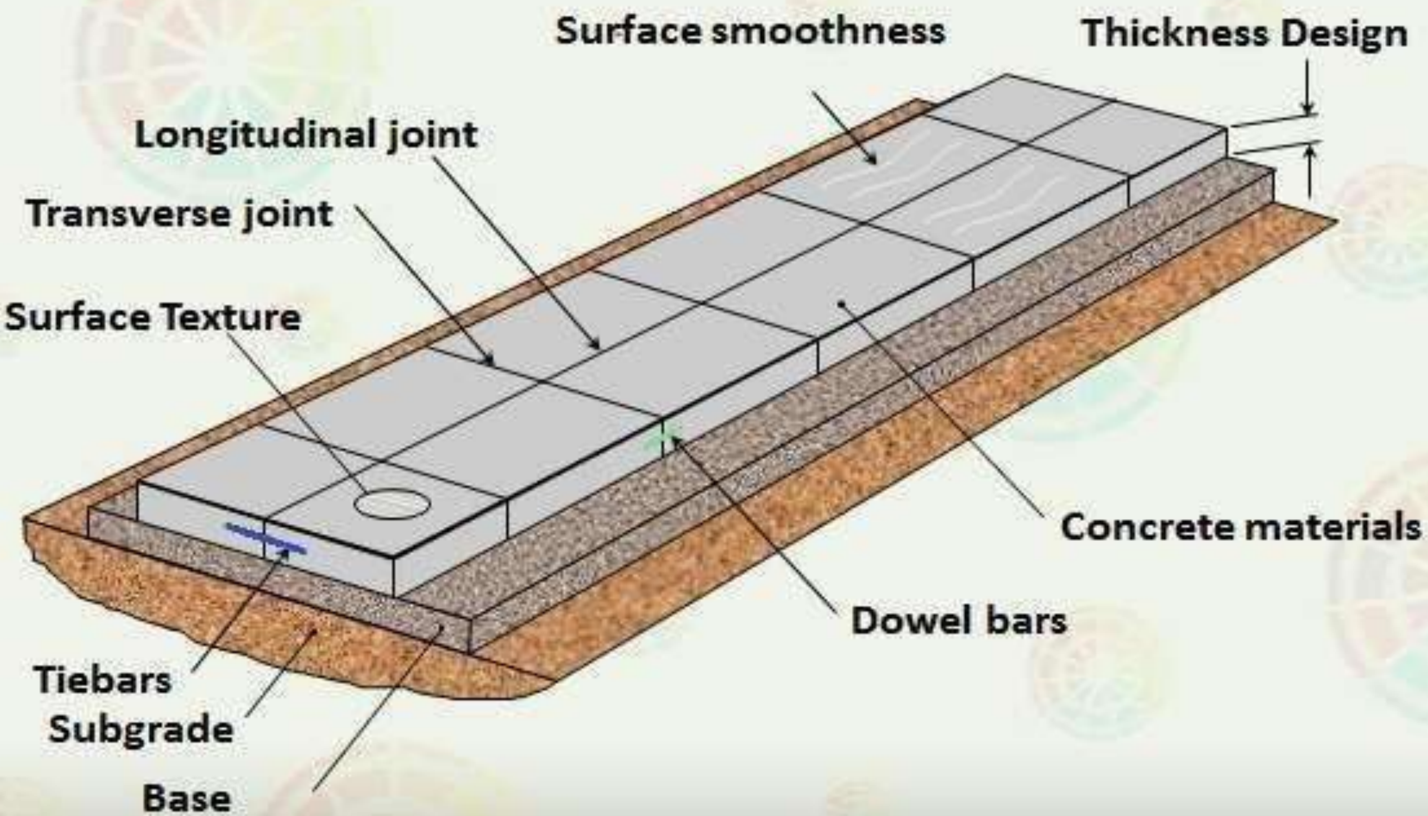
- soil subgrade.
- drainage layer.
- sub-base for ‘dry lean concrete’(heavy Traffic Loads)
  - separation membrane laid on top of base course.
- CC pavement slabs Using ‘ paving quality concrete’ (PQC)
- construction of different types of joints in CC pavements.

DLC, which is popularly used for the construction of sub-base layer under concrete pavement in India, is a zero slump concrete with maximum aggregate– cement **ratio** of 15:1. The water content of **DLC** varies from 5.5% to 7% of total dry weight of the mixture.

The separation membrane between concrete pavement slab and sub-base has the following functions:
(i) **Reducing the frictional forces** between concrete slab and sub-base and helps to movement of concrete slab
(ii) It **prevents the loss of cement and water** in immature concrete which significantly affects the strength and durability of hardened concrete. (iii)It **avoids the mixing** up of sub-base materials and freshly placed concrete.

Polythene sheeting of 125 microns which is a waterproof material, is commonly used as separation membrane

Basic Components of a Concrete Pavement



Types of Rigid Pavements

Jointed **Plain Concrete** Pavements does not use any reinforcing steel

Jointed **Reinforced Concrete** Pavements Reinforcing steel placed at mid height and discontinued at the joints

Continuously Reinforced Pavements This method is very costly and generally not used in India.

Pre-Stressed Concrete Pavement(Comprises new and innovative construction methods. Among these Plain CC pavements are most commonly used eg: Fibre Reinforced Concrete Pavements)

Equipments required for the different phases of concrete road construction :

- Three wheeled or vibratory roller for compaction purpose(bottom layers)

- Shovels, spades and Sieving screens
- Concrete mixer for mixing of concrete
- Formwork and iron stakes

- Watering devices - Water Lorries, water carriers or watering cans
- Wooden hand tampers for concrete compaction
- Cycle pump/pneumatic air blower for cleaning of joint

- Mild steel sections and blocks for making joint grooves for finishing purpose

METHODS OF CONSTRUCTION OF CC PAVEMENTS :

1. Construction by Slip Form Paver
2. Construction by Fixed Form Paver.
3.Construction by Fixed Form and labour oriented method of paving.

Basically different operation involved in construction of CC pavements slabs are :

a) spreading prepared concrete mix to desired thickness, grade and cross profiles.
b) Compacting.
c) Finishing the surface to desired surface profile.
d) Texturing.
e) Curing
f) Cutting of construction joints and longitudinal joints.

Site Preparation

Before construction begins, the construction site must be carefully prepared, This includes preparing the grade or road base, sub grade and sub base-

- First the site is graded to cut high points and fill low areas to the desired roadway profile elevations.
- Generally, cut material can be used as embankment fill.
- A course of material is placed on the sub grade to provide drainage and stability.
- A course of fairly rigid material, sometimes cement- or asphalt treated, that is placed on the sub base to provide a stable platform for the concrete pavement slab.

Construction by slip form paver

- Slip form paving equipment is capable of spreading fresh concrete from central mixing plant
- **Guided wires** are suitably fixed along with both sides of pavement slab with vertical tolerance of +/- 2mm, and lateral tolerance +/- 10mm
- Paver equipped with electric sensor to pave the slab upto required thickness and profile.
- Paver moves forward at speed of 1.5m per minute and concreting compaction, floating and finishing are completed.
- As stiff mix is fed into machine, paver moves forward, edges of slipformed slab remains in position and does not slump.

- Surface is textured using rectangular steel wire brush
- Then resin-based aluminized liquid is sprayed on surface and sides and left to cure for 8 to 12 hours.

- **Contraction and longitudinal joints** are cut and marked.

- Curing is further continued by covering pavement surface and sides by 2 to 3 layers of **moist hessian** for about 3 hours. The **hessian** is kept moist for a minimum curing period of 14 days.

- GOMACO GP-2400 Concrete Slipform Paver(720P_HD).mp4

SLIP FORM PAVER

Construction by Fixed Form Paver

- The fixed form paving train shall consist of separate powered machines which spread, compact and finish the concrete in a continuousoperation.

- The concrete is discharged without segregation into a hopper spreader which is equipped with means for controlling its rate of deposition on to thesub-base.

- The spreader is operated to strike off concrete up to a level requiring a small amount of cutting down by the distributor of the spreader.

- The distributor of spreader strikes off the concrete to the surcharge adequate to ensure that the vibratory compactor thoroughly compacts the layer. If necessary, poker vibrators shall be used adjacent to the side forms and edges of the previously constructedslab.
- The vibratory compactor is set to strike off the surface slightly high so that it is cut down to the required level by the oscillating beam.

- The final finisher finishes the surface to the required level and smoothness as specified, care being taken to avoid bringing up of excessive mortar to the surface by over working.

- Narrow grooves of specified width and depth are cut using diamond saw machine to provide **transverse contraction joints and longitudinal joints**

- Further curing is done by covering the pavement surface and sides by 2 to 3 layers of moist hessian for about 3 hours. The hessian is kept moist for a minimum curing period of 14 days.

- Neha Infra Concrete Fixed Form Paver(720P_HD).mp4

fix form

Construction by Fixed Form and labour oriented method of paving.

- This method is used when CC pavements is to be constructed in short stretches of narrow road using small machinery.
- Steel side forms are fixed in position and exact position of dowel and tie bars marked.
- Concrete is placed between side forms with surcharge and compaction and levelling is done by vibrating screeds resting on side forms.
- Irregularities are corrected by adding or removing concrete , followed by compaction and finishing.
- Then surface is textured manually using steel brush with long handle .Curing compound is sprayed by hand using a pressure sprayer.

➢ After CC is set , side forms are removed and shifted forward; curing compound is applied on sides of slab. Location of Contraction and longitudinal joints are cut and marked.

- Further curing is done by covering the pavement surface and sides by 2 to 3 layers of moist hessian for about 3 hours. The hessian is kept moist for a minimum curing period of 14 days

TYPES OF JOINTS IN CC PAVEMENTS:

A)Longitudinal joints (parallel to traffic flow).

B)Transverse joints (perpendicular to traffic flow) Contraction joints Expansion joints. construction joints.

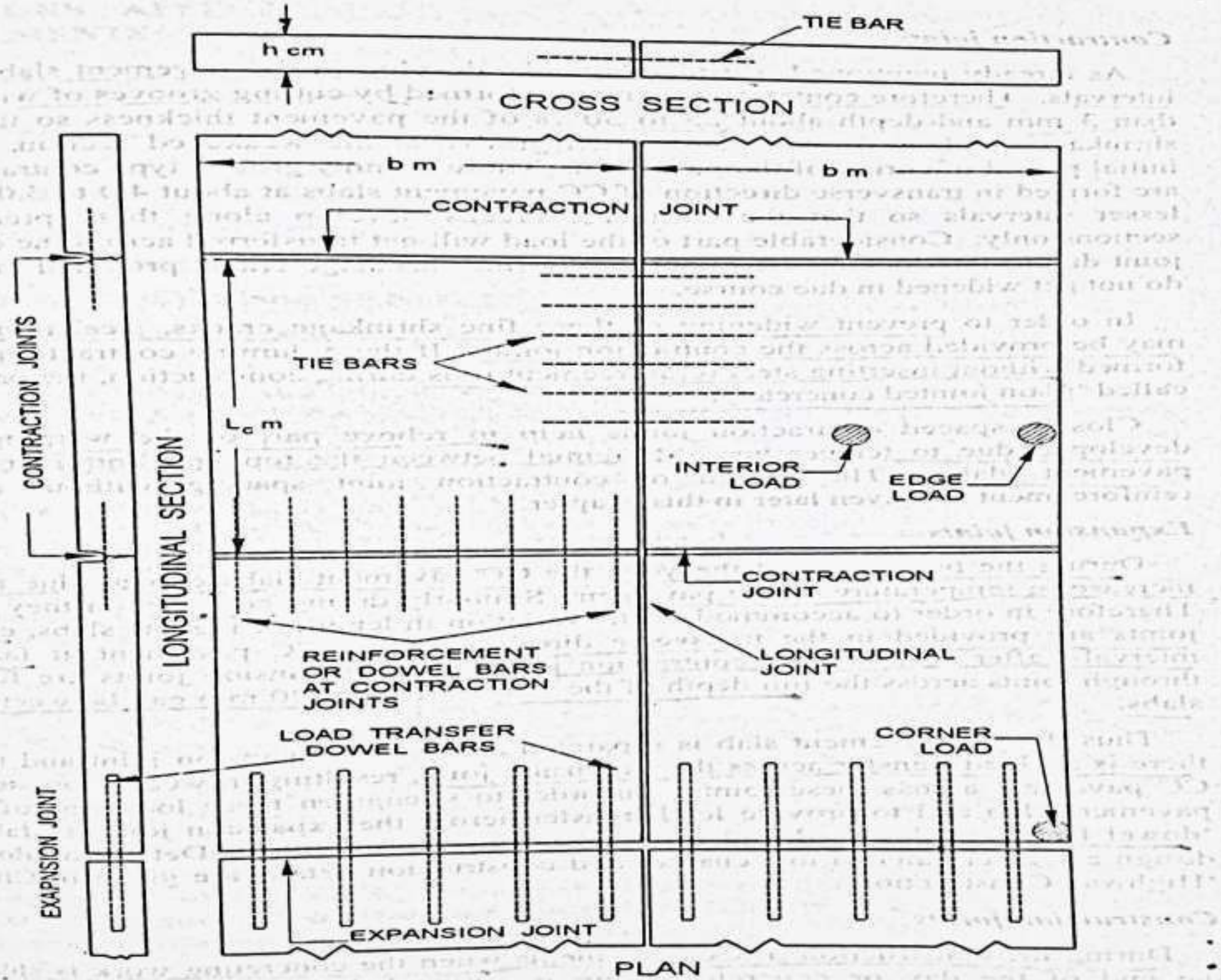


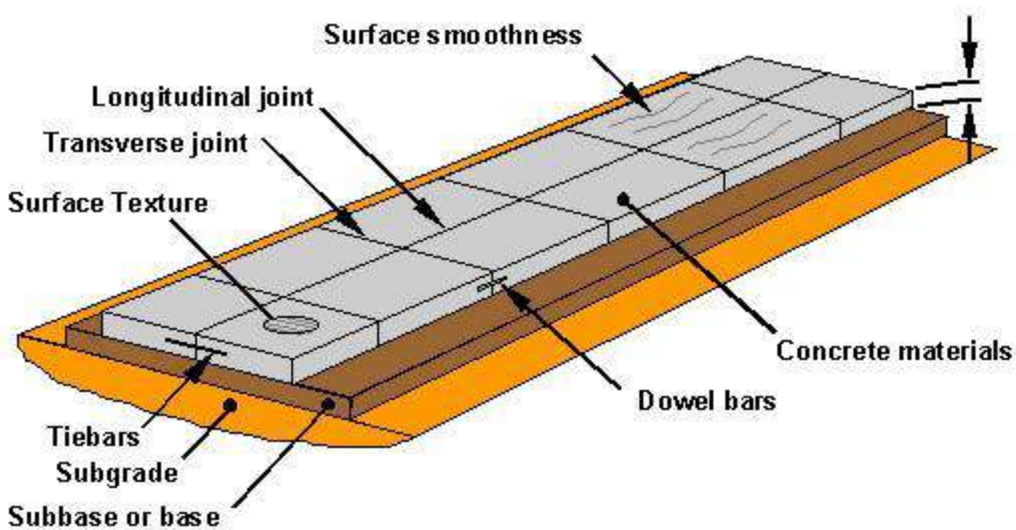
Fig. 7.17 Layout of different joints in CC pavements

Longitudinaljoints

- Pavements of width more than **4.5M,** there is a need to provide a Longitudinal Joints

- During initial period of curing ,shrinkage cracks usually develops in CC pavements ,when length or width of the slabs exceeds 4.5 to 5 m width or more.

- Hence longitudinal joints are provided whose spacing depends on width of traffic lane . For instance if width is 3.5 or3.75 , then spacing of longitudinal



joints is also 3.5 or 3.75m respectively.

Transverse joints

CONTRACTION JOINTS

- During initial period of curing ,shrinkage cracks usually develops in CC pavements ,when length or width of the slabs exceeds 4.5 to 5 m width or more.
- Contraction joints are dummy joints introduced by cutting narrow grooves about **one-fourth to one- third** the slab thickness ➢ Final shrinkages can develop only these grooves only.

- Prevent the formation of irregular cracks due to restraint in free contraction of concrete **.**

Two methods of construction of contraction joints:

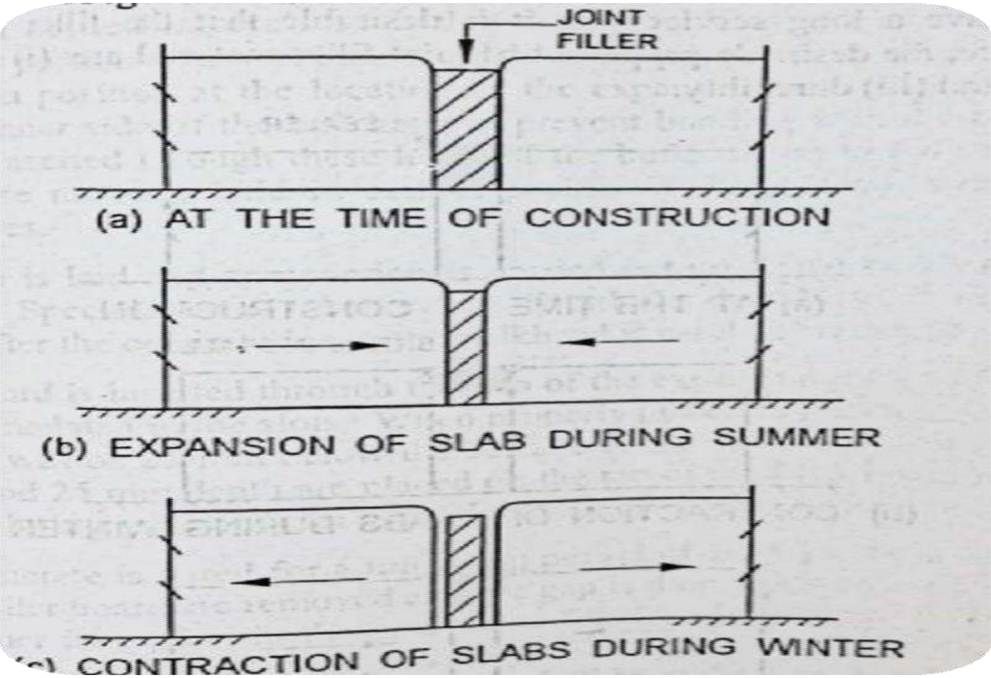
1. Joints without dowel bars’.

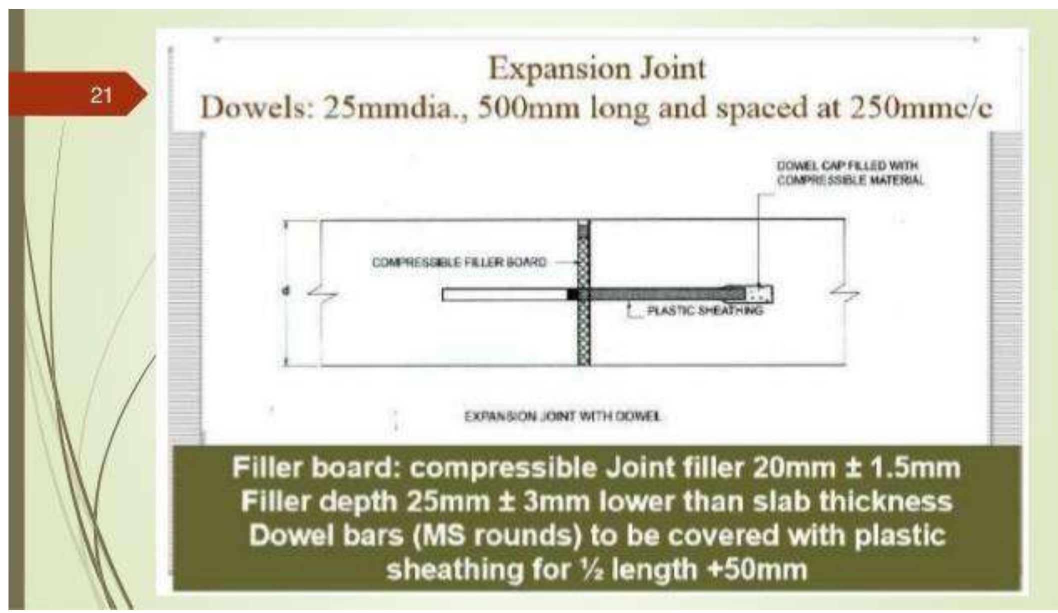
2. With dowel bars.

contraction joint saw cutting of CC pavement_PQC j(720P_HD).mp4

Expansion joints

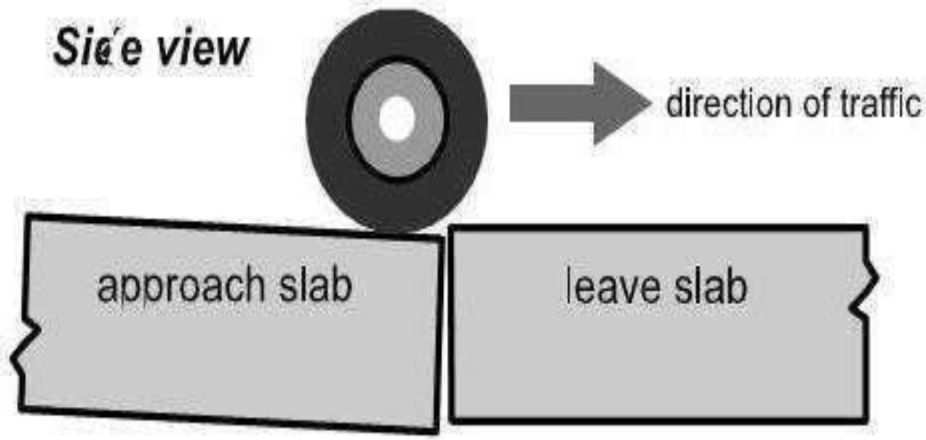
- Expansion joints are through transverse joints with gap of 20-25mm between two slabs to prevent expansion during summer season
  - Expansion joints are provided at intervals of 60-120M.
- Steel dowel bars are installed at expansion joints to facilitate load transfer from one slab to another slab, during moment of heavy wheel loads.
- They are allows expansion of slabs due to temperature.
- A joint filler board of compressible material is used to fill the gap between the adjacent slabs at the joint.
- The joint groove is filled by a sealant .
- **Rubberized bitumen** is commonly used

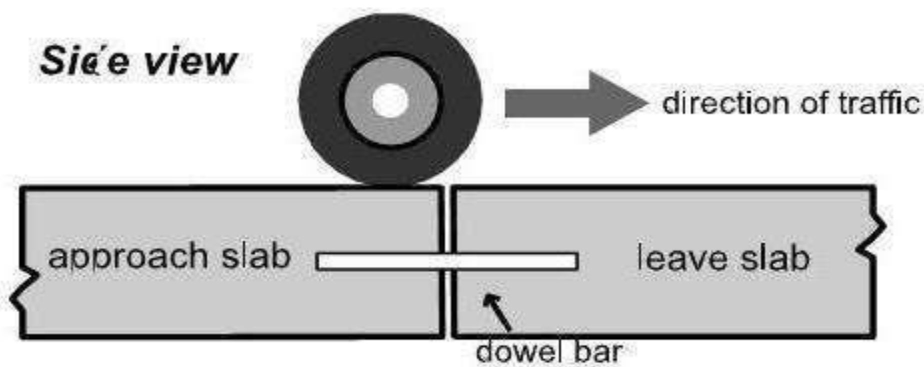

JOINT FILLER
(a) AT THE TIME OF CONSTRUCTION
(b) EXPANSION OF SLAB DURING SUMMER
(c) CONTRACTION OF SLABS DURING WINTER


21
Expansion Joint
Dowels: 25mmdia., 500mm long and spaced at 250mmc/c
DOWEL CAP FILLED WITH COMPRESSIBLE MATERIAL
COMPRESSIBLE FILLER BOARD
d
PLASTIC SHEATHING
EXPANSION JOINT WITH DOWEL
Filler board: compressible Joint filler 20mm ± 1.5mm
Filler depth 25mm ± 3mm lower than slab thickness
Dowel bars (MS rounds) to be covered with plastic
sheathing for ½ length +50mm

Construction Joints

- These joints are provided at locations where concrete construction operations are stopped after the day's work.
- Dowel bars are introduced in construction joints for load transfer.
- It is desirable to merge construction joints with expansion joints.


Side view
direction of traffic
approach slab
leave slab


Side view
direction of traffic
approach slab
leave slab
dowel bar

LOAD DISTRIBUTION

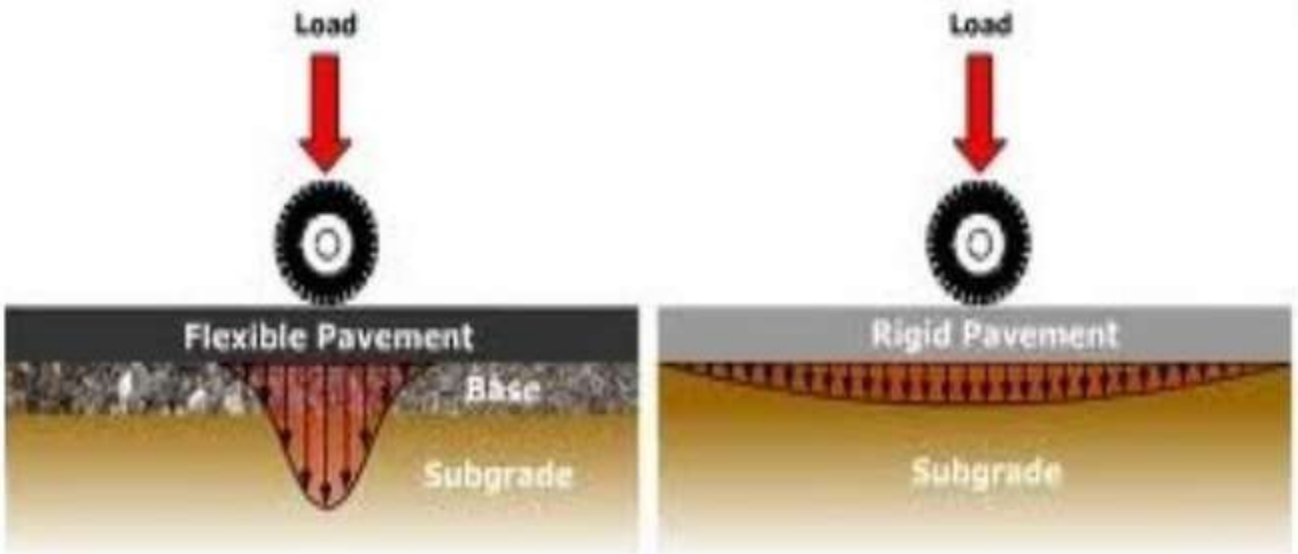


Figure 1: Rigid and Flexible Pavement Load Distribution

OPENING TO TRAFFIC

The entire surface of newly laid pavement is carefully examined for :

1. Fine cracks have developed on surface
2. Non-uniform settlements of CC slabs has taken place near abutments or along high embankments.

If any such defect is noticed ,then corrective measures may be taken up.

A newly constructed CC pavement stretchshall be opened to traffic only after a minimum curing of 28 days.

Thank you...