EINSTEIN
COLLEGE OF ENGINEERING
Sir. C.V. Raman Nagar, Tirunelveli-12

Department of Civil Engineering
CE – 46 HIGHWAY ENGINEERING

Lecture notes

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Aim: The objective of the course is to educate the students on the various components of Highway Engineering. It exposes the students to highway planning, engineering surveys for highway alignment, Design of Geometric Elements of Highways and Urban roads

Objectives:

- Introduction to Transportation Systems Engineering
- Explaining Transportation Planning
- Geometric Design of highways
- Pavement Design of highways
- Explaining Traffic Engineering

Text Book(s):


Reference Book(s):

4. Specifications for Road and Bridges, MORTH (India)
UNIT 1

Road classification

The roads can be classified in many ways. The classification based on speed and accessibility is the most generic one. Note that as the accessibility of road increases, the speed reduces. (See figure 1). Accordingly, the roads can be classified as follows in the order of increased accessibility and reduced speeds.

- Freeways: Freeways are access-controlled divided highways. Most freeways are four lanes, two lanes each direction, but many freeways widen to incorporate more lanes as they enter urban areas. Access is controlled through the use of interchanges, and the type of interchange depends upon the kind of intersecting road way (rural roads, another freeway etc.)

- Expressways: They are superior type of highways and are designed for high speeds (120 km/hr is common), high traffic volume and safety. They are generally provided with grade separations at intersections. Parking, loading and unloading of goods and pedestrian traffic is not allowed on expressways.

- Highways: They represent the superior type of roads in the country. Highways are of two types - rural highways and urban highways. Rural highways are those passing through rural areas (villages) and urban highways are those passing through large cities and towns, ie. urban areas.

- Arterials: It is a general term denoting a street primarily meant for through traffic usually on a continuous route. They are generally divided highways with fully or partially controlled access. Parking, loading and unloading activities are usually restricted and regulated. Pedestrians are allowed to cross only at intersections/designated pedestrian crossings.

- Local streets: A local street is the one which is primarily intended for access to residence, business or abutting property. It does not normally carry large volume of traffic and also it allows unrestricted parking and pedestrian movements.
Collector streets: These are streets intended for collecting and distributing traffic to and from local streets and also for providing access to arterial streets. Normally full access is provided on these streets. There are few parking restrictions except during peak hours.

**Figure 1:** Speed vs accessibility

### FACTORS AFFECTING HIGHWAY ALIGNMENT

**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.

Design speed is defined as the highest continuous speed at which individual vehicles can travel with safety on the highway when weather conditions are conducive. Design speed is different from the legal speed limit which is the speed limit imposed to curb a common tendency of drivers to travel beyond an accepted safe speed. Design speed is also different from the desired speed which is the maximum speed at which a driver would travel when unconstrained by either traffic or local geometry.
Since there are wide variations in the speed adopted by different drivers, and by different types of vehicles, design speed should be selected such that it satisfy nearly all drivers. At the same time, a higher design speed has cascading effect in other geometric designs and thereby cost escalation. Therefore, an 85th percentile design speed is normally adopted. This speed is defined as that speed which is greater than the speed of 85% of drivers. In some countries this is as high as 95 to 98 percentile speed.

**Topography**

The next important factor that affects the geometric design is the topography. It is easier to construct roads with required standards for a plain terrain. However, for a given design speed, the construction cost increases multiform with the gradient and the terrain. Therefore, geometric design standards are different for different terrain to keep the cost of construction and time of construction under control. This is characterized by sharper curves and steeper gradients.

**Other factors**

In addition to design speed and topography, there are various other factors that affect the geometric design and they are briefly discussed below:

- **Vehicle:** The dimensions, weight of the axle and operating characteristics of a vehicle influence the design aspects such as width of the pavement, radii of the curve, clearances, parking geometrics etc. A *design vehicle* which has standard
weight, dimensions and operating characteristics are used to establish highway design controls to accommodate vehicles of a designated type.

- **Human**: The important human factors that influence geometric design are the physical, mental and psychological characteristics of the driver and pedestrians like the reaction time.

- **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. The geometric design is thus based on this design volume, capacity etc.

- **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.

**Nagpur classification**

In Nagpur road classification, all roads were classified into five categories as National highways, State highways, Major district roads, Other district roads and village roads.

**National highways**

- They are main highways running through the length and breadth of India connecting major ports, foreign highways, capitals of large states and large industrial and tourist centers including roads required for strategic movements.
- It was recommended by Jayakar committee that the National highways should be the frame on which the entire road communication should be based.
- All the national highways are assigned the respective numbers.
- For e.g. the highway connecting Delhi-Ambala-Amritsar is denoted as NH-1 (Delhi-Amritsar), where as a bifurcation of this highway beyond Fullundar to Srinagar and Uri is denoted as NH-1_A.
- They are constructed and maintained by CPWD.
- The total length of National highway in the country is 58,112 Kms, and constitute about 2% of total road networks of India and carry 40% of total traffic.
State highways

- They are the arterial roads of a state, connecting up with the national highways of adjacent states, district head quarters and important cities within the state
- They also serve as main arteries to and from district roads.
- Total length of all SH in the country is 1,37,119 Kms.

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 kms of MDR.

Other district roads

- Roads serving rural areas of production and providing them with outlet to market centers or other important roads like MDR or SH.

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 kms of ODR+VR out of the total 33,15,231 kms of all type of roads.

Roads classification criteria

Apart from the classification given by the different plans, roads were also classified based on some other criteria. They are given in detail below.

Based on usage

This classification is based on whether the roads can be used during different seasons of the year.

- All-weather roads: Those roads which are negotiable during all weathers, except at major river crossings where interruption of traffic is permissible up to a certain extent are called all weather roads.
- Fair-weather roads: Roads which are negotiable only during fair weather are called fair weather roads.

Based on carriage way

This classification is based on the type of the carriage way or the road pavement.

- Paved roads with hards surface : If they are provided with a hard pavement course such roads are called paved roads.(eg: stones, Water bound macadam (WBM), Bituminous macadam (BM), concrete roads)
• Unpaved roads: Roads which are not provided with a hard course of at least a WBM layer they is called unpaved roads. Thus earth and gravel roads come under this category.

**Alignment**

The position or the layout of the central line of the highway on the ground is called the alignment. Horizontal alignment includes straight and curved paths. Vertical alignment includes level and gradients. Alignment decision is important because a bad alignment will enhance the construction, maintenance and vehicle operating costs. Once an alignment is fixed and constructed, it is not easy to change it due to increase in cost of adjoining land and construction of costly structures by the roadside.

**Requirements**

The requirements of an ideal alignment are

• The alignment between two terminal stations should be short and as far as possible be straight, but due to some practical considerations deviations may be needed.
• The alignment should be easy to construct and maintain. It should be easy for the operation of vehicles. So to the maximum extend easy gradients and curves should be provided.
• It should be safe both from the construction and operating point of view especially at slopes, embankments, and cutting. It should have safe geometric features.
• The alignment should be economical and it can be considered so only when the initial cost, maintenance cost, and operating cost are minimum.

**Factors controlling alignment**

We have seen the requirements of an alignment. But it is not always possible to satisfy all these requirements. Hence we have to make a judicial choice considering all the factors.

The various factors that control the alignment are as follows:

• Obligatory points: These are the control points governing the highway alignment. These points are classified into two categories. Points through which it should pass and points through which it should not pass. Some of the examples are:
  • Bridge site: The bridge can be located only where the river has straight and permanent path and also where the abutment and pier can be strongly founded. The road approach to the bridge should not be curved and skew crossing should be avoided as possible. Thus to locate a bridge the highway alignment may be changed.
o Mountain: While the alignment passes through a mountain, the various alternatives are to either construct a tunnel or to go round the hills. The suitability of the alternative depends on factors like topography, site conditions and construction and operation cost.

o Intermediate town: The alignment may be slightly deviated to connect an intermediate town or village nearby.

These were some of the obligatory points through which the alignment should pass. Coming to the second category, that is the points through which the alignment should not pass are:

- 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.

Traffic: The alignment should suit the traffic requirements. Based on the origin-destination data of the area, the desire lines should be drawn. The new alignment should be drawn keeping in view the desire lines, traffic flow pattern etc. Geometric design: Geometric design factors such as gradient, radius of curve, sight distance etc. also govern the alignment of the highway. To keep the radius of curve minimum, it may be required to change the alignment. The alignments should be finalized such that the obstructions to visibility do not restrict the minimum requirements of sight distance. The design standards vary with the class of road and the terrain and accordingly the highway should be aligned. Economy: The alignment finalized should be economical. All the three costs i.e. construction, maintenance, and operating cost should be minimum. The construction cost can be decreased much if it is possible to maintain a balance between cutting and filling. Also try to avoid very high embankments and very deep cuttings as the construction cost will be very higher in these cases.

Cross sectional elements

Overview

The features of the cross-section of the pavement influences the life of the pavement as well as the riding comfort and safety. Of these, pavement surface characteristics affect both of these. Camber, kerbs, and geometry of various cross-sectional elements are important aspects to be considered in this regard. They are explained briefly in this chapter.
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 \( 1 \text{ in } n \) 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 1. The common types of camber are parabolic, straight, or combination of them (Figure 1).

![Different Types of Camber](image)

**Figure 1:** Different types of camber

<table>
<thead>
<tr>
<th>Surface type</th>
<th>IRC Value Heavy</th>
<th>IRC Value Light</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete/Bituminous</td>
<td>2 %</td>
<td>1.7 %</td>
</tr>
<tr>
<td>Gravel/WBM</td>
<td>3 %</td>
<td>2.5 %</td>
</tr>
<tr>
<td>Earthen</td>
<td>4 %</td>
<td>3.0 %</td>
</tr>
</tbody>
</table>

*Table 1: IRC Values for camber*
Width of carriage way

Width of the carriage way or the width of the pavement depends on the width of the traffic lane and number of lanes.

Width of a traffic lane depends on the width of the vehicle and the clearance. Side clearance improves operating speed and safety.

The maximum permissible width of a vehicle is $2.44$ m 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 (Figure 1a). However, the side clearance required is about $0.53$ m, on either side or $1.06$ m in the center.

Therefore, a two lane road require minimum of $3.5$ meter for each lane (Figure 1b). The desirable carriage way width recommended by IRC is given in Table 1.

<table>
<thead>
<tr>
<th>Table 1: IRC Specification for carriage way width</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single lane</td>
</tr>
<tr>
<td>Two lane, no kerbs</td>
</tr>
<tr>
<td>Two lane, raised kerbs</td>
</tr>
<tr>
<td>Intermediate carriage</td>
</tr>
<tr>
<td>Multi-lane</td>
</tr>
</tbody>
</table>

Figure 1: Lane width for single and two lane roads
UNIT 2

Kerbs

Kerbs indicate the boundary between the carriage way and the shoulder or islands or footpaths. Different types of kerbs are (Figure 1):

- **Low or mountable kerbs**: This type of kerbs are provided such that they encourage the traffic to remain in the through traffic lanes and also allow the driver to enter the shoulder area with little difficulty. The height of this kerb is about 10 cm above the pavement edge with a slope which allows the vehicle to climb easily. This is usually provided at medians and channelization schemes and also helps in longitudinal drainage.

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

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

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

*Figure 1*: Different types of kerbs

**Shoulders**

Shoulders are provided along the road edge and are intended for accommodation of stopped vehicles, serve as an emergency lane for vehicles and provide lateral support for base and surface courses. The shoulder should...
be strong enough to bear the weight of a fully loaded truck even in wet conditions. The shoulder width should be adequate for giving working space around a stopped vehicle. It is desirable to have a width of 4.6 m for the shoulders. A minimum width of 2.5 m is recommended for 2-lane rural highways in India.

**Parking lanes**

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

**Bus-bays**

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

**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. These roads are provided to avoid congestion in the expressways and also the speed of the traffic in those lanes is not reduced.

**Drainage**

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

**Footpath**

Footpaths are exclusive right of way to pedestrians, especially in urban areas. They are provided for the safety of the pedestrians when both the pedestrian traffic and vehicular traffic is high. Minimum width is 1.5 meter and may be increased based on the traffic. The footpath should be either as smooth as the pavement or more smoother than that to induce the pedestrian to use the footpath.

**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. To prevent ribbon
development along highways, control lines and building lines may be provided. Control line is a line which represents the nearest limits of future uncontrolled building activity in relation to a road. Building line represents a line on either side of the road, between which and the road no building activity is permitted at all. The right of way width is governed by:

- Width of formation: It depends on the category of the highway and width of roadway and road margins.
- Height of embankment or depth of cutting: It is governed by the topography and the vertical alignment.
- Side slopes of embankment or cutting: It depends on the height of the slope, soil type etc.
- Drainage system and their size which depends on rainfall, topography etc.
- Sight distance considerations: On curves etc. there is restriction to the visibility on the inner side of the curve due to the presence of some obstructions like building structures etc.
- Reserve land for future widening: Some land has to be acquired in advance anticipating future developments like widening of the road.

<table>
<thead>
<tr>
<th>Road classification</th>
<th>Roadway width in m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plain and</td>
</tr>
<tr>
<td></td>
<td>Mountainous and</td>
</tr>
<tr>
<td></td>
<td>rolling terrain</td>
</tr>
<tr>
<td></td>
<td>steep terrain</td>
</tr>
<tr>
<td>Open areas</td>
<td></td>
</tr>
<tr>
<td>NH/SH</td>
<td>45</td>
</tr>
<tr>
<td>MDR</td>
<td>25</td>
</tr>
<tr>
<td>ODR</td>
<td>15</td>
</tr>
<tr>
<td>VR</td>
<td>12</td>
</tr>
<tr>
<td>Built-up areas</td>
<td></td>
</tr>
<tr>
<td>NH/SH</td>
<td>30</td>
</tr>
<tr>
<td>MDR</td>
<td>20</td>
</tr>
<tr>
<td>ODR</td>
<td>15</td>
</tr>
<tr>
<td>VR</td>
<td>10</td>
</tr>
</tbody>
</table>
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 1. A typical cross section of a ROW is given in Figure 1.

Overview

Horizontal alignment is one of the most important features influencing the efficiency and safety of a highway. A poor design will result in lower speeds and resultant reduction in highway performance in terms of safety and comfort. In addition, it may increase the cost of vehicle operations and lower the highway capacity. 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 setback distance. These will be discussed in this chapter and the following two chapters.

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 1. 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 2.
Table 1: Terrain classification

<table>
<thead>
<tr>
<th>Terrain classification</th>
<th>Cross slope (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain</td>
<td>0-10</td>
</tr>
<tr>
<td>Rolling</td>
<td>10-25</td>
</tr>
<tr>
<td>Mountainous</td>
<td>25-60</td>
</tr>
<tr>
<td>Steep</td>
<td>&gt; 60</td>
</tr>
</tbody>
</table>

The recommended design speed is given in Table 2.

Table 2: Design speed in km/hr as per IRC (ruling and minimum)

<table>
<thead>
<tr>
<th>Type</th>
<th>Plain</th>
<th>Rolling</th>
<th>Hilly</th>
<th>Steep</th>
</tr>
</thead>
<tbody>
<tr>
<td>NS&amp;SH</td>
<td>100-80</td>
<td>80-65</td>
<td>50-40</td>
<td>40-30</td>
</tr>
<tr>
<td>MDR</td>
<td>80-65</td>
<td>65-50</td>
<td>40-30</td>
<td>30-20</td>
</tr>
<tr>
<td>ODR</td>
<td>65-50</td>
<td>50-40</td>
<td>30-25</td>
<td>25-20</td>
</tr>
<tr>
<td>VR</td>
<td>50-40</td>
<td>40-35</td>
<td>25-20</td>
<td>25-20</td>
</tr>
</tbody>
</table>

**Horizontal curve**

The presence of horizontal curve imparts centrifugal force which is a reactive force acting outward on a vehicle negotiating it. Centrifugal force depends on speed and radius of the horizontal curve and is counteracted to a certain extent by transverse friction between the tyre and pavement surface. On a curved road, this force tends to cause the vehicle to overrun or to 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 figure 1.

![Figure 1: Effect of horizontal curve](image-url)
They are the centrifugal force \( P \) acting outward, weight of the vehicle \( (W) \) acting downward, and the reaction of the ground on the wheels \( (R_A \text{ and } R_B) \). 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 is given by

\[
P = \frac{Wv^2}{gR}
\]

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

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

\[
\frac{P}{W} = \frac{v^2}{gR}
\]

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

\[
Ph = Wb \frac{b}{2} \text{ or } \frac{P}{W} = \frac{b}{2h}
\]

At the equilibrium over turning is possible when

\[
\frac{v^2}{gR} = \frac{b}{2h}
\]

and for safety the following condition must satisfy:

\[
\frac{b}{2h} > \frac{v^2}{gR}
\]
The second tendency of the vehicle is for transverse skidding. i.e. When the centrifugal force $P$ is greater than the maximum possible transverse skid resistance due to friction between the pavement surface and tyre. The transverse skid resistance ($F$) is given by:

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

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

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

At equilibrium, when skidding takes place (from equation 1)

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

and for safety the following condition must satisfy:

$$f > \frac{v^2}{gR} \quad (3)$$

Equation 2 and 3 give the stable condition for design. If equation 2 is violated, the vehicle will overturn at the horizontal curve and if equation 3 is violated, the vehicle will skid at the horizontal.

**curve 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 figure 1.
Forces acting on a vehicle on horizontal curve of radius $R \text{ m}$ at a speed of $\nu \text{ m/sec}^2$ are:

- $F$ the centrifugal force acting horizontally outwards through the center of gravity,
- $W$ the weight of the vehicle acting downwards 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,

\[
\begin{align*}
P \cos \theta &= W \sin \theta + F_A + F_B \\
&= W \sin \theta + f(R_A + R_B) \\
&= W \sin \theta + f(W \cos \theta + P \sin \theta)
\end{align*}
\]

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

\[
\frac{P \cos \theta}{W \cos \theta} = \frac{W \sin \theta}{W \cos \theta} + \frac{fW \cos \theta}{W \cos \theta} + \frac{fP \sin \theta}{W \cos \theta}
\]
\[ \frac{P}{W} = \tan \theta + f + f \frac{P}{W} \tan \theta \]

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

\[ \frac{P}{W} = \frac{\tan \theta + f}{1 - f \tan \theta} \tag{1} \]

We have already derived an expression for \( \frac{P}{W} \). By substituting this in equation 1, we get:

\[ \frac{v^2}{gR} = \frac{\tan \theta + f}{1 - f \tan \theta} \tag{2} \]

This is an exact expression for superelevation. But normally, \( f = 0.15 \) and \( \theta < 4^\circ \), \( 1 - f \tan \theta \approx 1 \) and for small \( \theta \), \( \tan \theta \approx \sin \theta = E/B = \epsilon \), then equation 2 becomes:

\[ \epsilon + f = \frac{v^2}{gR} \tag{3} \]

where, \( \epsilon \) is the rate of super elevation, \( f \) the coefficient of lateral friction 0.15, \( v \) the speed of the vehicle in \( \text{m/sec}^2 \), \( g \) the radius of the curve in \( \text{m} \) and \( \epsilon \) in \( \text{m/sec}^2 \).

Three specific cases that can arise from equation 3 are as follows:

1. If there is no friction due to some practical reasons, then \( f = 0 \) and equation 3 becomes \( \epsilon = \frac{v^2}{gR} \). This results in the situation where the pressure on the outer and inner wheels are same; requiring very high super-elevation \( \epsilon \).

2. If there is no super-elevation provided due to some practical reasons, then \( \epsilon = 0 \) and equation 3 becomes \( f = \frac{v^2}{gR} \). This results in a very high coefficient of friction.
If \( f = 0.15 \) and \( \epsilon = 0 \), then for safe traveling speed from equation 3 is given by
\[
v_b = \sqrt{f g R}
\]
where \( v_b \) is the restricted speed.

**Design of super-elevation**

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

**Step 1**

Find \( \epsilon \) for 75 percent of design speed, neglecting \( f \), i.e.
\[
e_1 = \frac{(0.75v)^2}{gR}
\]

**Step 2**

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

**Step 3**

Find \( f_1 \) for the design speed and max \( \epsilon \), i.e.
\[
f_1 = \frac{v^2}{gR} - \epsilon = \frac{v^2}{gR} - 0.07
\]
If \( f_1 < 0.15 \), then the maximum \( \epsilon = 0.07 \) is safe for the design speed, else go to step 4.

**Step 4**

Find the allowable speed \( v_a \) for the maximum \( \epsilon = 0.07 \) and \( f = 0.15 \)
\[
v_a = \sqrt{0.22gR}
\]
If \( v_a \geq v \) then the design is adequate, otherwise use speed adopt control measures or look for speed control measures.

**Maximum and minimum super-elevation**

Depends on (a) slow moving vehicle and (b) heavy loaded trucks with high CG. IRC specifies a maximum super-elevation of 7 percent for plain and rolling terrain, while that of hilly terrain is 10 percent and urban road is 4 percent. The minimum super elevation is 2-4 percent for drainage purpose, especially for large radius of the horizontal curve.

**Attainment of super-elevation**

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

2. Rotation of the pavement cross section to attain full super elevation by: There are two methods of attaining super elevation by rotating the pavement
   1. Rotation about the center line: The pavement is rotated such that the inner edge is depressed and the outer edge is raised both by half the total amount of super elevation, i.e., by \( \frac{E}{2} \) with respect to the centre.
   2. Rotation about the inner edge: Here the pavement is rotated raising the outer edge as well as the centre such that the outer edge is raised by the full amount of super elevation with respect to the inner edge.

**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 super elevation 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 super elevation and coefficient of friction.

\[
R_{ruling} = \frac{v^2}{g(e + f)} \tag{1}
\]

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.

**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.
UNIT-3

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 \textit{(tangent point)} to the desired radius of the circular curve at the other end \textit{(curve point)}. There are five objectives for providing transition curve and are given below:

1. to introduce gradually the centrifugal force between the tangent point and the beginning of the circular curve, avoiding sudden jerk on the vehicle. This increases the comfort of passengers.
2. to enable the driver turn the steering gradually for his own comfort and security,
3. to provide gradual introduction of super elevation, and
4. to provide gradual introduction of extra widening.
5. to enhance the aesthetic appearance of the road.

Type of transition curve

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

1. it fulfills the requirement of an ideal transition curve, that is;
   1. rate of change of centrifugal acceleration is consistent (smooth) and
   2. radius of the transition curve is at the straight edge and changes to at the curve point \( \frac{1}{R} \) and calculation and field implementation is very easy.

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. 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:

\[
\begin{align*}
    c &= \frac{v^2}{R} - \frac{0}{t}, \\
    &= \frac{v^2}{R \cdot \frac{t}{v}},
\end{align*}
\]
Therefore, the length of the transition curve in meters is

\[ L_{s_1} = \frac{v^3}{cR}, \]

where \( c \) is the rate of change of centrifugal acceleration given by an empirical formula suggested by IRC as below:

\[ c = \frac{80}{75 + 3.6v^2}, \]

subject to:

\[ c_{\text{min}} = 0.5, \]
\[ c_{\text{max}} = 0.8. \]

2. Rate of introduction of super-elevation

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

\[ E = \varepsilon B = \varepsilon(W + W_e). \]

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 is:

\[ L_{s_2} = N\varepsilon(W + W_e) \]

3. By empirical formula

IRC suggest the length of the transition curve is minimum for a plain and rolling terrain:

\[ L_{s_3} = \frac{35v^2}{R} \]

and for steep and hilly terrain is:

\[ L_{s_3} = \frac{12.96v^2}{R} \]
and the shift \( s \) as:

\[
s = \frac{L_s^2}{24R}
\]  \( (6) \)

The length of the transition curve \( L_s \) is the maximum of equations 1, 3 and 4 or 5, i.e.

\[
L_s = \text{Max} : \{L_{s1}, L_{s2}, L_{s3}\}
\]  \( (7) \)

**Case (a)**

For single lane roads:

\[
\alpha = \frac{s}{R} \text{ radians}
\]

\[
= \frac{180s}{\pi R} \text{ degrees}
\]

\[
\alpha/2 = \frac{180s}{2\pi R} \text{ degrees}
\]  \( (1) \)

Therefore,

\[
m = R - R \cos \left( \frac{\alpha}{2} \right)
\]  \( (2) \)
Figure 1: Set-back for single lane roads \( L_s < L_c \)

For multi lane roads, if \( d \) is the distance between centerline of the road and the centerline of the inner lane, then

\[
m = R - (R - d) \cos \left( \frac{180s}{2\pi(R - d)} \right) \tag{3}
\]

\[
m = R - R \cos \left( \frac{\alpha}{2} \right) \tag{4}
\]
\( L_s < L_c \)

**Case (a)**

For single lane roads:

\[
\alpha = \frac{s}{R} \text{ radians} \\
= \frac{180s}{\pi R} \text{ degrees} \\
\alpha/2 = \frac{180s}{2\pi R} \text{ degrees}
\]

Therefore,

\[
m = R - R \cos \left( \frac{\alpha}{2} \right)
\]

**Figure 1:** Set-back for single lane roads \( L_s < L_c \)

For multi lane roads, if \( d \) is the distance between centerline of the road and the centerline of the inner lane, then

\[
m = R - (R - d) \cos \left( \frac{180s}{2\pi(R - d)} \right)
\]
Vertical alignment-I

Overview

The vertical alignment of a road consists of gradients (straight lines in a vertical plane) and vertical curves. The vertical alignment is usually drawn as a profile, which is a graph with elevation as vertical axis and the horizontal distance along the centre line of the road as the horizontal axis. Just as a circular curve is used to connect horizontal straight stretches of road, vertical curves connect two gradients. When these two curves meet, they form either convex or concave. The former is called a summit curve, while the latter is called a valley curve. This section covers a discussion on gradient and summit curves.

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 for designing the vertical curve. Before finalizing the gradients, the construction cost, vehicular operation cost and the practical problems in the site also has to be considered. Usually steep gradients are avoided as far as possible because of the difficulty to climb and increase in the construction cost. More about gradients are discussed below.

Effect of gradient

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

**Representation of gradient**

The positive gradient or the ascending gradient is denoted as \( +n \) and the negative
gradient as \( -n \). The deviation angle \( \mathcal{N} \) is: when two grades meet, the angle which
measures the change of direction and is given by the algebraic difference between the two
grades

\[
(n_1 - (-n_2)) = n_1 + n_2 = \alpha_1 + \alpha_2
\]

Example: 1 in 30 = 3.33% \( \approx 2^o \) is a steep gradient, while 1 in 50 = 2% \( \approx 1^o10' \) is a flatter gradient. The gradient
representation is illustrated in the figure 1.

\[\text{Figure 1: Representation of gradient}\]

**Types of gradient**

Many studies have shown that gradient up to 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 as flat as two percent is adopted. Although,
flatter gradients are desirable, it is evident that the cost of construction will also be very
high. Therefore, IRC has specified the desirable gradients for each terrain. However, it
may not be economically viable to adopt such gradients in certain locations, steeper
grades are permitted for short duration. Different types of grades are discussed below
and the recommended type of gradients for each type of terrain and type of gradient is
given in table 1.

<table>
<thead>
<tr>
<th>Terrain</th>
<th>Ruling</th>
<th>Limitings</th>
<th>Exceptional</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain/Rolling</td>
<td>3.3</td>
<td>5.0</td>
<td>6.7</td>
</tr>
<tr>
<td>Hilly</td>
<td>5.0</td>
<td>6.0</td>
<td>7.0</td>
</tr>
</tbody>
</table>
Ruling gradient, limiting gradient, exceptional gradient and minimum gradient are some types of gradients which are discussed below.

**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 flatter terrain, it may be possible to provide flat gradients, but in hilly terrain it is not economical and sometimes not possible also. The ruling gradient is adopted by the designer by considering a particular speed as the design speed and for a design vehicle with standard dimensions. But our country has a heterogeneous traffic and hence it is not possible to lay down precise standards for the country as a whole. Hence IRC has recommended some values for ruling gradient for different types of terrain.

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

**Exceptional gradient**
Exceptional gradient are very steeper gradients given at unavoidable situations. They should be limited for short stretches not exceeding about 100 metres at a stretch. In mountainous and steep terrain, successive exceptional gradients must be separated by a minimum 100 metre length gentler gradient. At hairpin bends, the gradient is restricted to 2.5%.

**Critical length of the grade**
The maximum length of the ascending gradient which a loaded truck can operate without undue reduction in speed is called critical length of the grade. A speed of 25 kmph is a reasonable value. This value depends on the size, power, load, grad-ability of the truck, initial speed, final desirable minimum speed

**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 require some slope for smooth flow of water. Therefore minimum gradient is provided for drainage purpose and it depends on the rain fall, type of soil and other site conditions. A minimum of 1 in 500 may be sufficient for concrete drain and 1 in 200 for open soil drains are found to give satisfactory performance.
UNIT 4

Summit curve

Summit curves are vertical curves with gradient upwards. They are formed when two gradients meet as illustrated in figure 1 in any of the following four ways:

1. when a positive gradient meets another positive gradient [figure 1a].
2. when positive gradient meets a flat gradient [figure 1b].
3. when an ascending gradient meets a descending gradient [figure 1c].
4. when a descending gradient meets another descending gradient [figure 1d].

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. Although a circular curve offers equal sight distance at every point on the curve, for very small deviation angles a circular curve and parabolic curves are almost congruent. Furthermore, the use of parabolic curves were found to give excellent riding comfort.

Design Consideration

In determining the type and length of the vertical curve, the design considerations are comfort and security of the driver, and the appearance of the profile alignment. Among these, sight distance requirements for the safety are most important on summit curves. The stopping sight distance or absolute minimum sight distance should be provided on these curves and where overtaking is not prohibited, overtaking sight distance or intermediate sight distance should be provided as far as possible. When a fast moving vehicle travels along a summit curve, there is less discomfort to the passengers. This is because the centrifugal force will be acting upwards while the vehicle negotiates a summit curve which is against the gravity and hence a part of the tyre pressure is relieved. Also if the curve is provided with adequate sight distance, the length would be sufficient to ease the shock due to change in gradient. Circular summit curves are identical since the radius remains same throughout and hence the sight distance.
this point of view, transition curves are not desirable since it has varying radius and so the sight distance will also vary. The deviation angles provided on summit curves for highways are very large, and so the simple parabola is almost congruent to a circular arc, between the same tangent points. Parabolic curves are easy for computation and also it had been found out that it provides good riding comfort to the drivers. It is also easy for field implementation. Due to all these reasons, a simple parabolic curve is preferred as summit curve.

**Length of the summit curve**

The important design aspect of the summit curve is the determination of the length of the curve which is parabolic. As noted earlier, the length of the curve is guided by the sight distance consideration. That is, a driver should be able to stop his vehicle safely if there is an obstruction on the other side of the road. Equation of the parabola is given by \( y = ax^2 \), where \( a = \frac{N}{2L} \), where \( N \) is the deviation angle and \( L \) is the length of the

In deriving the length of the curve, two situations can arise depending on the uphill and downhill gradients when the length of the curve is greater than the sight distance and the length of the curve is greater than the sight distance.

Let \( L \) is the length of the summit curve, \( S \) is the SSD/ISD/OSD, \( N \) is the deviation angle, \( h_1 \) driver’s eye height (1.2 m), and \( h_2 \) the height of the obstruction, then the length of the summit curve can be derived for the following two cases. The length of the summit curve can be derived from the simple geometry as shown below:

**Case a. Length of summit curve greater than sight distance**

\( L > S \)

**Figure 1:** Length of summit curve \( L > S \)

The situation when the sight distance is less than the length of the curve is shown in figure 1.
Case b. Length of summit curve less than sight distance

The second case is illustrated in figure 1

\[ a = \frac{N}{2L} \]
\[ h_1 = aS_1^2 \]
\[ h_2 = aS_2^2 \]
\[ S_1 = \sqrt{\frac{h_1}{a}} \]
\[ S_2 = \sqrt{\frac{h_2}{a}} \]
\[ S_1 + S_2 = \sqrt{\frac{h_1}{a}} + \sqrt{\frac{h_2}{a}} \]
\[ S^2 = \left( \frac{1}{\sqrt{a}} \right)^2 \left( \sqrt{h_1} + \sqrt{h_2} \right)^2 \]
\[ S^2 = \frac{2L}{N} \left( \sqrt{h_1} + \sqrt{h_2} \right)^2 \]
\[ L = \frac{NS^2}{2 \left( \sqrt{h_1} + \sqrt{h_2} \right)^2} \]

(1)

Figure 1: Length of summit curve (\( L < S \))
From the basic geometry, one can write

$$S = \frac{L}{2} + \frac{h_1}{n_1} + \frac{h_2}{n_2} = \frac{L}{2} + \frac{h_1}{n_1} + \frac{h_2}{N-n_2}$$  \hspace{1cm} (1)

Therefore for a given $L$, $h_1$, and $h_2$ to get minimum $S$, differentiate the above equation with respect to $h_1$ and equate it to zero. Therefore,

$$\frac{dS}{dh_1} = -\frac{h_1}{n_1^2} + \frac{h_2}{N-n_1} = 0$$

$$0 = h_1 (N-n_1)^2 = h_2 n_1^2$$

$$h_1 (N^2 + n_1^2 - 2N n_1) = h_2 n_1^2$$

$$h_1 N^2 + h_1 n_1^2 - 2N n_1 h_1 = h_2 n_1^2$$

$$(h_2 - h_1) n_1^2 + 2N h_1 n_1 - h_1 N^2 = 0$$

Solving the quadratic equation for $n_1$,

$$n_1 = \frac{-2Nh_1 \pm \sqrt{(2Nh_1)^2 - 4(h_2-h_1)(-h_1 N^2)}}{2(h_2-h_1)}$$

$$= \frac{-2Nh_1 + \sqrt{4N^2 h_1^2 + 4h_1 N^2 h_2 - 4h_1^2 N^2}}{2(h_2-h_1)}$$

$$= \frac{-2Nh_1 + 2N\sqrt{h_1 h_2}}{2(h_2-h_1)}$$

$$n_1 = \frac{N \sqrt{h_1 h_2} - h_1 N}{h_2 - h_1}$$  \hspace{1cm} (2)

Now we can substitute $n_1$ back to get the value of minimum value of $L$ for a given $h_1$, $h_2$, $n_2$, and $n_1$. Therefore,

$$S = \frac{L}{2} + \frac{h_1}{N \sqrt{h_1 h_2} - h_1} + \frac{h_2}{N - \frac{N \sqrt{h_1 h_2} - h_1}{h_2 - h_1}}$$
Solving for $L$,

\[
L = \frac{L}{2} + \frac{h_1 (h_2 - h_1)}{N (\sqrt{h_1 h_2} - h_1)} + \frac{h_2 (h_2 - h_1)}{Nh_2 - Nh_1 - N\sqrt{h_1 h_2} + Nh_1}
\]

\[
= \frac{L}{2} + \frac{h_1 (h_2 - h_1)}{N (\sqrt{h_1 h_2} - h_1)} + \frac{h_2 (h_2 - h_1)}{N (h_2 - \sqrt{h_1 h_2})}
\]

\[
= \frac{L}{2} + \frac{(h_2 - h_1)(h_1 h_2 - h_1 \sqrt{h_1 h_2} + h_2 \sqrt{h_1 h_2} - h_1 h_2)}{N (\sqrt{h_1 h_2} - h_1)(h_2 - \sqrt{h_1 h_2})}
\]

\[
= \frac{L}{2} + \frac{(h_2 - h_1)(\sqrt{h_1 h_2} (h_2 - h_1))}{N (h_2\sqrt{h_1 h_2} - h_1 h_2 + h_1 \sqrt{h_1 h_2} - h_1 h_2)}
\]

\[
= \frac{L}{2} + \frac{(h_2 - h_1)\sqrt{h_1 h_2} (\sqrt{h_2 + \sqrt{h_1}}) (\sqrt{h_2 - \sqrt{h_1}})}{N \sqrt{h_1 h_2} (h_2 - 2\sqrt{h_1 h_2} + h_2)}
\]

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

\[
= \frac{L}{2} + \frac{(h_2 - h_1) (\sqrt{h_2} + \sqrt{h_1})}{N (\sqrt{h_2} - \sqrt{h_1})}
\]

\[
= \frac{L}{2} + \frac{(\sqrt{h_2} + \sqrt{h_1}) (\sqrt{h_2} - \sqrt{h_1}) (\sqrt{h_2} + \sqrt{h_1})}{N (\sqrt{h_2} - \sqrt{h_1})}
\]

\[
= \frac{L}{2} + \frac{(\sqrt{h_2} + \sqrt{h_1})^2}{N}
\]

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

\[
L = 2S - \frac{(\sqrt{2h_1} + \sqrt{2h_2})^2}{N}
\]
When stopping sight distance is considered the height of driver's eye above the road surface \( h_1 \) is taken as 1.2 metres, and height of object above the pavement surface \( h_2 \) is taken as 0.15 metres. If overtaking sight distance is considered, then the value of driver's eye height \( h_1 \) and the height of the obstruction \( h_2 \) are taken equal as 1.2 metres.

Valley curve

Valley curve or sag curves are vertical curves with convexity downwards. They are formed when two gradients meet as illustrated in figure 1 in any of the following four ways:

1. when a descending gradient meets another descending gradient [figure 1a].
2. when a descending gradient meets a flat gradient [figure 1b].
3. when a descending gradient meets an ascending gradient [figure 1c].
4. when an ascending gradient meets another ascending gradient [figure

**Design considerations**

There is no restriction to sight distance at valley curves during day time. But visibility is reduced during night. In the absence or inadequacy of street light, the only source for visibility is with the help of headlights. Hence valley curves are designed taking into account of headlight distance. In valley curves, the centrifugal force will be acting downwards along with the weight of the vehicle, and hence impact to the vehicle will be more. This will result in jerking of the vehicle and cause discomfort to the passengers. Thus the most important design factors considered in valley curves are: (1) impact-free...
movement of vehicles at design speed and (2) availability of stopping sight distance under headlight of vehicles for night driving.

For gradually introducing and increasing the centrifugal force acting downwards, the best shape that could be given for a valley curve is a transition curve. Cubic parabola is generally preferred in vertical valley curves. See figure 1.

During night, under headlight driving condition, sight distance reduces and availability of stopping sight distance under headlight is very important. The headlight sight distance should be at least equal to the stopping sight distance. There is no problem of overtaking sight distance at night since the other vehicles with headlights could be seen from a considerable distance.

**Length of the valley curve**

The valley curve is made fully transitional by providing two similar transition curves of equal length. The transitional curve is set out by a cubic parabola where

\[ y = bx^3 \]

\[ b = \frac{2N}{3L^2} \]

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 0.6m/sec^3.
2. **safety criteria**: that is the driver should have adequate headlight sight distance at any part of the country.

**Comfort criteria**

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

![Figure 1: Valley curve details](image-url)
For a cubic parabola, the Value Safety criteria
Length of the valley curve for headlight distance may be determined for two conditions: (1) length of the valley curve greater than stopping sight distance and (2) length of the valley curve less than the stopping sight distance. of $R$ for length $L_s$ is given by:

**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. The case is shown in figure 1.

![Figure 1: Valley curve, case 1](image)

From the geometry of the figure, we have:

$$h_1 + S \tan \alpha = aS^2$$
where $N$ is the deviation angle in radians, $h_1$ is the height of headlight beam, $\alpha$ is the head beam inclination in degrees and $S$ is the sight distance. The inclination $\alpha$ is $\approx 1$ degree.

**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 1.

From the figure,

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

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

(1)
\[ S > L \]

**Figure 1**: Valley curve, case 2,

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

1. Flexible pavement
2. Rigid pavement

**Flexible pavement:**

These pavements reflect the deformation of sub grade and the subsequent layers to the surface.

**Rigid pavement:**

The rigid characteristic of the pavement are associated with rigidity or flexural strength or slab action so the load is distributed over a wide area of sub grade soil.

**Flexible pavement: Definition**

These pavements reflect the deformation of sub grade and the subsequent layers to the surface. Flexible, usually asphalt, is laid with no reinforcement or with a specialized fabric reinforcement that permits limited flow or repositioning of the roadbed under ground changes.

- The design of flexible pavement is based on load distributing characteristic of the component layers. The black top pavement including water & gravel bound macadam fall in this category.

- Flexible pavement on the whole has low or negligible flexible strength flexible in their structural action). The flexible pavement layers transmit the vertical or compressive stresses to the lower layers by grain transfer through contact points of granular structure.

- The vertical compressive stress is maximum on the pavement surface directly under the wheel load and is equal to contact pressure under the wheels. Due to the ability to distribute the stress to large area in the shape of truncated cone the stresses get decreased in the lower layer.

- As such the flexible pavement may be constructed in a number of layers and the top layer has to be strongest as the highest compressive stresses.
To be sustained by this layer, in addition to wear and tear, the lower layer have to take up only lesser magnitude of stress as there is no direct wearing action die to traffic loads, therefore inferior material with lower cast can be used in the lower layers.

**Rigid pavement: Definition**

The rigid characteristic of the pavement are associated with rigidity or flexural strength or slab action so the load is distributed over a wide area of sub grade soil. Rigid pavement is laid in slabs with steel reinforcement.

The rigid pavements are made of cement concrete either plan, reinforced or prestressed concrete.

Critical condition of stress in the rigid pavement is the maximum flexural stress occurring in the slab due to wheel load and the temperature changes.

Rigid pavement is designed and analyzed by using the elastic theory.

**Advantages of Rigid Pavement**

1. Rigid lasts much, much longer i.e 30+ years compared to 5-10 years of flexible pavements.
2. In the long run it is about half the cost to install and maintain. But the initial costs are somewhat high.
3. Rigid pavement has the ability to bridge small imperfections in the subgrade.
5. High efficiency in terms of functionality

**Comparison of Flexible and Rigid Pavement**

1. Deformation in the sub grade is transferred to the upper layers
2. Design is based on load distributing characteristics of the component layers
3. Have low flexural strength
4. Load is transferred by grain to grain contact
5. Have low completion cost but repairing cost is high
6. Have low life span
7. Surfacings cannot be laid directly on the sub grade but a sub base is needed
8. Deformation in the sub grade is not transferred to subsequent layers
9. Design is based on flexural strength or slab action
10. Have high flexural strength
11. No such phenomenon of grain to grain load transfer exists
12. Have low repairing cost but completion cost is high
13. Life span is more as compared to flexible
14. Surfacing can be directly laid on the sub grade
15. Thermal stresses are more vulnerable
8. No thermal stresses are induced as the pavement have the ability to contract and expand freely
9. That's why expansion joints are not needed
10. Strength of the road is highly dependent on the strength of the sub grade
11. Rolling of the surfacing is needed
12. Road can be used for traffic within 24 hours
13. Force of friction is less
   Deformation in the sub grade is not transferred to the upper layers.

Mix design

Through mix design, suitable proportions of the ingredients (coarse aggregates, fine aggregates, cement, water and admixture, if any) are estimated, keeping in view the strength, workability, durability and economic considerations. These proportions are achieved through iterative experimental procedure in the laboratory. There are number of methods for mix design of cement concrete, and a detailed discussion can be obtained elsewhere (Neville and Brooks 1999).

Water-cement ratio is an important consideration in the mix design process. As water cement ratio is increased in concrete, the durability and strength decreases, however, the workability enhances. Depending on the type of construction, workability requirements are different.

For large scale production of cement concrete, the proportioning operation is performed in the batch mixing plant. Figure 3 shows a photograph of a typical concrete batch mixing plant.
Properties of fresh concrete

Ideally a fresh concrete should be workable, should not segregate or bleed during construction. Constituent properties, their proportions, aggregate shape and sizes, temperature affect the performance of fresh mix. The tests that are conducted on fresh concrete include workability test and air-content test. Some of tests through which workability of can be estimated are Kelly ball penetration test, slump test, compacting factor test, Vee bee test and flow table test etc.

Curing of concrete

Presence of adequate amount of moisture, at some requisite temperature and for a suitable period of time, is necessary to complete the hydration process of cement. This process is called curing. The curing conditions significantly affect the final strength achieved by the concrete. For pavement construction, only in-situ curing methods are applicable. Curing compounds are sometimes applied to retain the moisture against evaporation. For final curing of concrete pavements continuous ponding or moistened hessain/ gunny bags are used.

Properties of hardened concrete

Tests are conducted on hardened concrete to estimate properties like, compressive strength, tensile strength, modulus of rupture, elastic modulus, Poisson's ratio, creep and shrinkage performance, durability, thermal expansion coefficient etc. These parameters are of functions of aggregate type, shape and size, type and quantity of cement and admixtures incorporated, water cement ratio, curing, age etc.

Compressive strength of concrete is the failure compressive stress on cubical or cylindrical samples of concrete. Compressive strength of concrete is related to the combined effect of temperature and time, a parameter called maturity. Maturity of
Concrete is calculated as the time of curing (in hours), multiplied by the temperature, (in degrees) above some specified reference temperature. Various empirical relationships are suggested to obtain the various strength parameters of concrete (elastic modulus, tensile strength, bending strength etc.) from the compressive strength of concrete.

Direct tension test on concrete is performed by applying tension to the cylindrical or dome shaped samples of concrete. Indirect tension is applied to concrete samples by split cylinder test.

Modulus of rupture of concrete is estimated by measuring the maximum bending stress on concrete beam subjected to pure bending in static condition.

Fatigue test is generally performed subjecting the concrete beams with repetitive flexural loading. The more is the stress ratio (defined as the ratio between the bending stress applied to the modulus of rupture) the less is the fatigue life. The empirically derived fatigue equation by PCA (1974) is the following:

\[
\log(N_f) = 11.78 - 12.11 \times SR \\
\text{and} \\
\log(N_f) > 5.725 
\]

\[
\text{Where, } N_f \text{ is the number of load applications to failure, } SR \text{ is the stress ratio with reference to 90 days modulus of rupture.}
\]

The equation suggested by AASHTO (1993) is the following:

\[
N_f = 23440(SR)^{2.21} 
\]

Transportation of concrete

The transportation of concrete is to be done in such a way that segregation and premature setting is avoided. Wheel barrow, truck mixer, dumper truck, belt conveyor, pipe-line etc. are the various ways concrete is transported to the construction site. Figure 4 shows a typical truck concrete mixer.

EMERGING ROAD MATERIALS

Modification of Existing Materials

Existing materials may require modifications so as to

- improve engineering properties of material
- satisfy general specification requirement of locally available material which in turn would prove to be cost effective
• meet the demand of special purpose materials having specific properties. Example: high or low permeability, enhanced shear strength etc

These have been discussed further under two sections as,

• binder (bitumen) modification
• aggregate modification

Design parameters

The design parameters can be primarily divided into three categories, material, traffic and environmental parameters and are discussed in the following.

Material parameters

The elastic modulus, Poisson's ratio, fatigue life, modulus of rupture etc. are the engineering parameters used for the structural design of the concrete pavement. The input parameters are either found out experimentally or estimated from various recommendations provided in the design guidelines (ACI 2001, ASTM 2003, IS-456). Statistically suitable design value is to be adopted if there are variations in the input parameters (Chakroborty and Das 2003). The modulus of subgrade reaction \( k \) is generally used for characterization of subgrade strength. \( k \) in idealized model represents the spring constant of a dense liquid foundation. The \( k \) value is obtained by performing plate load test on the subgrade. The IRC:58 (2002) provides a table with suggested \( k \) values when CBR values of subgrade are known. Concrete slab, in general, are constructed on bound, stabilized, or unbound sub-base layer, and not directly on subgrade. The sub-base layer provided below the concrete pavement serves certain purposes (Austroads 2004), such as,

1. It provides uniform support to the concrete slab.
2. It limits pumping at joints and slab edges.
3. It controls shrinkage of concrete slab or swelling of subgrade soil

The effective \( k \) value that includes the sub-base layer and the subgrade should be used for design of thickness of the concrete slab. This effective \( k \) value can be obtained by performing plate load test on the constructed sub-base, or by computational means. Various guidelines (IRC:58 2002, AASHTO 1993, PCA 1984, Austroads 2004) suggest suitable values of effective \( k \) when the type and thickness of the sub-base layer is known.
UNIT-5

Traffic parameters

Design life

Design life is the number of years (or number of standard axle repetitions) for which the pavement is being designed. A pavement is expected to serve satisfactorily within the design life. For concrete pavements 20 to 40 years may be assumed as the design life (PCA 1984, IRC:58 2002).

Basic design principle

Though different approaches for concrete pavement design are suggested in various guidelines, the design principles tend to remain similar across different guidelines, for example, PCA (1994), Austroads (2004), NCHRP (2004), IRC:58 (2002) etc., except the AASHTO (1993) provisions, which is based on empirical approach. The basic steps involved in the design of concrete pavement method can be summarized as follows:

- The **developed stresses** due to load for a trial thickness of the concrete slab are calculated for various loading configuration and the critical one is chosen. The axle loads are generally divided into different axle load groups and the load stresses are calculated individually.
- The ratio between the load stress and **Modulus of Rupture** (MOR) is known as **stress ratio**. The stress ratio determines how many repetitions the pavement can sustain (i.e. **allowable traffic repetitions**) for the individual axle load group. If the stress ratio is 0.55 or lower, it can withstand virtually infinite number of traffic repetitions (PCA 1984).
- The ratio between the allowable repetitions to the **expected traffic repetitions** is the **damage fraction**. The calculation process is repeated for various axle loads (sometimes, for various seasons, or various timings of the day), and the sum of individual damage fractions (cumulative fatigue damage) should be less than equal to one for pavement design being safe. If found unsafe, the trial thickness is changed and the design process is repeated.
- The design process may also include considerations for temperature stress, moisture stresses, and erosion distress.

A brief discussion on concrete pavement design approaches suggested by various design practices, viz. PCA method (1984), Austroads method (2004), AASHTO method (1993), NCHRP mechanistic-empirical method (2004) and the Indian Roads Congress (IRC) method (2002) are placed in the following:
Portland Cement Association (PCA) Method

The PCA method is based on Westergaard, Picket and Ray's work and further theoretical analysis by Finite Element Method (Huang 1993). The data used to develop the PCA method is generated from various road tests, like, ASSHO road test, Arlington test (conducted by PCA), Bates test road, and Maryland road test (PCA 1984). The PCA design method is based on the following two considerations (PCA 1984):

- The fatigue damage on the concrete slab, due to repetitive application of traffic load is estimated. The cumulative fatigue damage principle is used to estimate the design thickness of the slab. Edge stress between the mid-way of the transverse joint is taken as critical configuration.
- The stresses due to warping and curling (due to temperature and moisture gradient) are not considered in the fatigue analysis as per PCA recommendation, because, most of the time the stresses generated are subtractive to the load stress.
- The possibility of erosion of pavement materials placed below the concrete slab is evaluated. The rate at which the slab is deflected due to axle load is used as a criterion for erosion. Theoretically, it can be shown that a thin pavement with smaller deflection basin is subjected to faster rate of deflection, compared to thicker slab. Hence thin slab is more susceptible to erosion. In a similar way the cumulative erosion damage is calculated for individual axle load groups. If this value is greater than one, then the design needs to be revised.

Austroads method

Austroads method has been adopted from PCA (1984) approach with modifications suited to Australian conditions (Austroads 2004). A bound mix or lean cement concrete is used as sub-base material. For a given CBR value of the subgrade and given thickness of cemented sub-base, the effective subgrade strength can be obtained from the chart provided. Figure-19 schematically shows such a chart. For different levels of traffic, Austroads (2004) suggests minimum values of the base thicknesses to be provided.
As per Austroads (2004), the concrete pavement slab thickness, for a given expected traffic repetitions, is designed considering the (i) flexural fatigue of the concrete slab and the (ii) subgrade erosion arising out of repeated deflections. For both the considerations, equations are suggested to calculate the allowable traffic repetitions. If the allowable traffic is less than the expected traffic, the design is revised by increasing the slab thickness. The concrete shoulders adopted are of integral type or structural type.

AASHTO method

The AASHTO method (1993) for design of concrete pavement has evolved from the AASHO road test (AASHO 1962). The AASHTO pavement design follows an empirical approach. Pavement performance in terms of present serviceability index (PSI), loss of serviceability, sub-grade and sub-base strength, cumulative traffic, properties of concrete, joint load transfer efficiency, drainage condition, overall standard deviation and reliability are the input parameters considered in the pavement design. The PSI value of the fresh pavement is assumed as 4.5 and the pavement is deemed to have failed when the PSI value reaches 2.5. The resilient moduli of the subgrade and sub-base materials are determined in the laboratory simulating the seasonal moisture content and stress situation. Suggested values are also available for given moisture content, plasticity index etc. The composite modulus of sub-grade reaction \(k\) is estimated from modulus of sub-grade reaction and elastic modulus of sub-base for various seasons and the depth of rigid foundation and the thickness of the sub-base. Design equation as well nomographs are available to estimate the slab thickness \(D\) from these input parameters.

NCHRP mechanistic-empirical (M-E) method
The NCHRP (2004) has recently developed concrete pavement design procedure based on mechanistic-empirical (M-E) approach. This approach attempts to reduce the extent of empiricism prevalent in the existing AASHTO (1993) guidelines. This proposed NCHRP pavement design system is modular in nature, that is, the design approach can be modified by parts (as and when new knowledge is available) without disrupting the overall design procedure. This approach also can take care of various wheel-axle load configuration (Khanum et al. 2004). As per this approach trial thickness of the slab is first assumed, and the stress, strain and displacement values are obtained. From these values, the performance of the pavement in terms of distresses (such as faulting, cracking) and smoothness and predicted. If these predicted performance parameters does not satisfy the required performance for a given reliability, the design revised. The design approach includes a large data-base as input parameters, for example, average daily traffic, traffic growth rate, traffic composition, hourly weather data on air temperature, precipitation, wind speed, percentage sunshine, relative humidity, pavement material engineering parameters, ground water depth, infiltration, drainage, hydraulic conductivity, thermal conductivity, heat capacity etc. The temperature stress is considered in this method, but the temperature profile is linearized to enhance computational efficiency (NCHRP 2004).

Indian Roads Congress (IRC) method

The Indian Roads Congress (IRC) guidelines, IRC:58 (2002), has adopted the Westergaard's equation to estimate load stress and Brdabury's equation to estimate temperature stress. The load stress is highest at the corner of the slab, lesser in edge and least in the interior. The order of variation of temperature stress is just the reverse of this. As per IRC:58 (2002), it is recommended that the design needs to be done for edge stress condition and subsequently check needs to be performed for corner stress condition so as to finalize the design. The following are the steps followed as per IRC:58 (2002) guideline for the design of concrete pavement:

- The input parameters are obtained to formulate the design problem. The joint spacing and the slab dimensions are decided. If there is a bound sub-base layer over the subgrade, a suitable value of effective $k$ is to be adopted.
- A trial thickness of the concrete slab is assumed.
- The edge stress is estimated for various axle loads from the given charts. Figure-20 schematically shows such a chart. The cumulative fatigue damage principle for fatigue is applied to check the adequacy of the slab thickness.
- The sum of edge stress due to load for the highest axle load group and the temperature stress should be less than the MOR of concrete, otherwise the design is revised.
- The adequacy of corner stress is checked with respect to MOR value and accordingly the design is finalized. Westergaard's corner stress formula is for
estimation of corner stress due to load, and the corner stress due to temperature is assumed to be zero.

![Schematic diagram of IRC:58 chart for estimation of load stress at the edge (IRC:58 2002)](image)

**Figure-20** Schematic diagram of IRC:58 chart for estimation of load stress at the edge (IRC:58 2002)

**Estimation of layer thicknesses**

- The thickness of the pavement is adjusted in such a way that the stress/strain developed is less than the allowable values obtained from past performance information.
- The two major modes of structural failure of pavement are **fatigue and rutting**.
  - **Fatigue**: Traffic applies repetitive load to the pavement surface, and the cracks start from bottom the bound layer/slab and propagate upwards. When the extent of surface cracks reaches a predefined level, the pavement is said to have failed due to flexural fatigue.

**Functional design**

The functional pavement design involves considerations of skid resistance, roughness, surface distresses, reflectivity of pavement surface etc. The functional pavement design considers mainly the surface features of a pavement.

**Drainage design**

A road needs to be designed in such a way that the rain/snow precipitation is drained off the pavement and its surroundings. A suitable surface drainage system for the pavement is designed for this purpose. Some water, however, will percolate into the pavement from...
its top surface and needs to be taken out of the pavement - this is done by providing an internal drainage system to the pavement. Water will also try to enter into the pavement from bottom due to capillary rise or due to rise in water table. A suitably designed sub-surface drainage system tries to avoid such a problem.

**Pavement thickness design charts**

For the design of pavements to carry traffic in the range of 1 to 10 msa, use chart 1 and for traffic in the range 10 to 150 msa, use chart 2 of IRC:37 2001. The design curves relate pavement thickness to the cumulative number of standard axles to be carried over the design life for different sub-grade CBR values ranging from 2 % to 10 %. The design charts will give the total thickness of the pavement for the above inputs. The total thickness consists of granular sub-base, granular base and bituminous surfacing. The individual layers are designed based on the the recommendations given below and the subsequent tables.

**Pavement composition**

**Sub-base**

Sub-base materials comprise natural sand, gravel, laterite, brick metal, crushed stone or combinations thereof meeting the prescribed grading and physical requirements. The sub-base material should have a minimum CBR of 20 % and 30 % for traffic upto 2 msa and traffic exceeding 2 msa respectively. Sub-base usually consist of granular or WBM and the thickness should not be less than 150 mm for design traffic less than 10 msa and 200 mm for design traffic of 1:0 msa and above.

**Base**

The recommended designs are for unbounded granular bases which comprise conventional water bound macadam (WBM) or wet mix macadam (WMM) or equivalent confirming to MOST specifications. The materials should be of good quality with minimum thickness of 225 mm for traffic up to 2 msa an 150 mm for traffic exceeding 2 msa.

**Bituminous surfacing**

The surfacing consists of a wearing course or a binder course plus wearing course. The most commonly used wearing courses are surface dressing, open graded premix carpet, mix seal surfacing, semi-dense bituminous concrete and bituminous concrete. For binder course, MOST specifies, it is desirable to use bituminous macadam (BM) for traffic upto 0.5 msa and dense bituminous macadam (DBM) for traffic more than 5 msa.
Numerical example

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

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

Solution

1. Distribution factor = 0.75

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

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

- The horizontal tensile stress/strain at the bottom of bound layer (bituminous surfacing, cemented base or concrete slab, as the case may be) is used as the governing parameter for fatigue failure.
- Conventionally, for design of concrete pavement stress is used as parameter, and for design of bituminous pavement strain is used as parameter.
- **Rutting**: Rutting is the accumulation of permanent deformation. This is the manifestation of gradual densification of pavement layers, and shear displacement of the subgrade.
The vertical strains on the pavement layers, mainly the vertical strain on the subgrade is assumed to be governing factor for rutting failure.

The rutting issue is not considered for concrete pavement design, because it does not have any permanent deformation.

- The fatigue/ rutting equations are developed from field or laboratory studies, where fatigue / rutting lives are obtained with respect to respective stress/ strain for fatigue/ rutting. For a given design life, thus, allowable fatigue and rutting stress/ strains can be estimated using the fatigue/ rutting equations.
- The various other types of pavement failures could be shrinkage, thermal fatigue, top down cracking (for bituminous pavement) etc.

**Concrete pavement**

- Concrete pavement is, in general, consists of three layers, subgrade, base layer and the concrete slab.
Generally bound base layers are used for concrete pavement construction. As per Indian specification, some example of such base layers are Dry Lean Concrete (DLC), Roller Compacted Concrete (RCC) (IRC:15-2002).

The concrete slab is generally of M40 to M50 grade of concrete as per Indian specifications, and is called as paving quality concrete (PQC) (IRC:15-2002).

**Bituminous pavement**

- The subgrade is a compacted soil layer.
- The base and sub-base course could be made up of bound or unbound granular layer. As per Indian specifications (MORT&H 2001), some examples of base or sub-base layers are: Granular sub-base(GSB), Water Bound Macadam (WBM), Wet Mix Macadam (WMM) etc.
- The binder course is made up bituminous material. As per Indian specifications (MORT&H 2001), some examples of binder course are: Bituminous Macadam (BM), Dense Bituminous Macadam (DBM) etc.
- The wearing course is the top bituminous layer which is comes in contact to the vehicle tyre. Wearing course provides impermeability to the pavement surface against water percolation (Chakroborty and Das 2003). The binder course and wearing course together are called bituminous surfacing.

**IRC METHOD OF FLEXIBLE PAVEMENT MIX DESIGN**

These guidelines will apply to design of flexible pavements for Expressway, National Highways, State Highways, Major District Roads, and other categories of roads. Flexible pavements are considered to include the pavements which have bituminous surfacing and granular base and sub-base courses conforming to IRC/ MOST standards. These guidelines apply to new pavements.

**Design criteria**

The flexible pavements has been modeled as a three layer structure and stresses and strains at critical locations have been computed using the linear elastic model. To give proper consideration to the aspects of performance, the following three types of pavement distress resulting from repeated (cyclic) application of traffic loads are considered:

1. Vertical compressive strain at the top of the sub-grade which can cause sub-grade deformation resulting in permanent deformation at the pavement surface.
2. Horizontal tensile strain or stress at the bottom of the bituminous layer which can cause fracture of the bituminous layer.
3. Pavement deformation within the bituminous layer.

While the permanent deformation within the bituminous layer can be controlled by meeting the mix design requirements, thickness of granular and bituminous layers are selected using the analytical design approach so that strains at the critical points are
within the allowable limits. For calculating tensile strains at the bottom of the bituminous layer, the stiffness of dense bituminous macadam (DBM) layer with 60/70 bitumen has been used in the analysis.

**Failure Criteria**

**Critical locations in pavement**

A and B are the critical locations for tensile strains $(\varepsilon_t)$. Maximum value of the strain is adopted for design. C is the critical location for the vertical subgrade strain $(\varepsilon_z)$ since the maximum value of the 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

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

(1)

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 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

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

(2)

$N_r$ 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.