Chapter: One

Geometric Design of Highways

1.1 Introduction

A geometric design is defined as the design of visible components of a highway. Therefore, the geometric designer can be considered as the architect of the roadway. The basic features of a highway are the carriageway itself, expressed in terms of the number of lanes used, the central reservation or median strip and the shoulders (including verges), horizontal and vertical alignments, intersections and the length of acceleration and deceleration lanes and so on. These components are highly influenced by characteristics of driver and vehicle performance. Depending on the level of the highway relative to the surrounding terrain, side-slopes may also be a design issue

In addition, radii of curves of an intersection are governed by the minimum radius of design vehicle being using the highway. Therefore, the purpose of a geometric design of the roadway is to provide a consistent design that satisfies the characteristics of driver, vehicle, pedestrians and safety.

1.2 Highway Design Control

Geometric design of highway is the determination of layout and features visible on highway. The emphasis is more on satisfying the need of the driver as well as to ensure the safety of the vehicle, the comfort while driving and efficiency. Other related factors are also considered based on the project.

Highway design depends on many factors, mostly include:

- Highway classification
- Traffic volume and traffic type
- Design Speed
- Design vehicle
- Cross-section of highway
- Presence of heavy vehicles on steep grades
- Topography and environmental
- Level of service
- Safety
- Funds
- Restrictions

1.2.1 Highway classification

Grouping streets and highways into systems or classes is called functional classification. The latter provides an identification of the role of any highway class in the highway network. Basically, roads can be classified (based on the area in which they locate) as Urban and Rural. Rural Roads are usually earth, dust and gravel surfaced. These roads are not demarcated or they do not have any carriage markings like that we see on urban roads. Usually rural roads do not have traffic engineering tools like give way signs, stop signs or traffic lights. The use of rural roads is designated to light vehicles or heavy vehicles alone, but will have users mixed at any time.

On the other hand, Urban roads are usually planned into an interconnected network owing to human and economic activities and exchange. These roads are usually associated with designated traffic engineering equipment like give way signs, stop signs and carriage markings. Moreover, Urban roads usually have demarcations of space of road, there are cases of separate cyclists ways, pedestrian ways and carriage ways for public buses.

In other words, in terms of population, Urban areas are those with a population of at least 5,000. They are further classified as urbanized area (with population of 50,000 or more) and small urban areas (with a population between 5,000 and 50,000).

Roads can be further classified into different types in accordance with their function which is generally related to the mobility and access they provide. Mobility and access are the two important objectives need to be achieved. Mobility means less interrupted follow of traffic, higher design speeds, less access points and sustain higher traffic volume. On the other hand, access indicates interrupted traffic, less design speed and higher accessibility.

In general, both urban and rural roads can be further classified into Freeway, arterials, collectors and locals as follows

- ***Freeway or Expressway:*** it is a type of road which is served only higher degree of mobility which means achieving higher design speeds and uninterrupted traffic flow. Therefore, freeways road have the highest design standards compared with other facilities. Figure 1.1 illustrates an example of freeway roads.

Figure 1.1: Freeway, Express or Motorway

- ***Arterial Road***: Theses Street are primarily for high traffic volume on a continuous road and it has a higher level of traffic mobility. In fact, this type of road has similar function of freeway with less degree of mobility. Figure 1.2 shows different types of arterial roads

Figure 1.2: a- Rural arterial road

Figure 1.2: b- urban arterial road

- ***Collector or distributer road***: collectors have good balancing between mobility and access. The main function of collectors is to facilitate the travel between the arterials and the local roads. They collect the traffic from local roads and convey it to the arterials roads. Furthermore, it has normally full access with a speed limit of 30-55Km/h. Figure 1.3 presents an example of collector road.

Figure 1.3: Collector road

- ***Local Street:*** this type is normally designed to provide access to different adjoining property (business, residential…. etc.) while minimizing speeds. Figure 1.4 illustrates an example of Local Street.

Figure 1.4: Local Street

1.2.2 Traffic volume and traffic type

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. The following types of traffic numbers are frequently used in highway design:

- ***Annual Average Daily Traffic (AADT)***: it is the total volume of vehicle traffic of a highway or road for a year divided by 365 days.
- ***Average Daily Traffic (ADT)***: it is the average number of vehicles using a roadway in a 24-hour period. ADTs can be calculated from any sample of repeated daily counts of traffic volumes, with duration as short as one week.
- ***Design Hourly Volume (DHV)***: it is the estimated number of vehicles using the roadway in the 30th highest hour of the year. This number is generally 8 to 12 percent of the ADT and is used extensively in determining lane widths and shoulder characteristics of the roadway cross section.
- ***Directional Design Hourly Volume (DDHV)***: the estimated number of vehicles traveling in one direction of a two-way roadway in the 30th highest hour of the year. This number must be at least 50 percent of the DHV and is usually in the range of 50 to 60 percent.

$DHV = ADT \times K$

$DDHV = DHV \times D$ or $DDHV = ADT \times K \times D$

where

DHV=design hourly volume.

DDHV=directional design hourly volume (vehicles per hour, veh/h).
ADT=average daily traffic (vehicles per day, veh/d).
K=design hourly volume factor (0.10 typically).
D=directional movement factor (0.60 typically).

Example 1: The following traffic counts were taken along an Arterial:

Day 1: 1900 vehicles

Day 2: 2150 vehicles, D=55%

Day 3: 2300 vehicles, K=12%

Day 4: 1950 vehicles

Day 5: 2000 vehicles

Find the ADT, DHV, and DDHV

ADT = (1900 + 2150 + 2300 + 1950 + 2000)/5 = 2060 veh /day

DHV = ADT x K = 2060 x 0.12 = 247 veh / hour

DDHV = DHV x D = 247 x 0.55 = 136 veh / hour in the peak direction

1.2.3 Design Speed

Design speed is the single most important factor that affects the geometric design. It directly affects the sight distance, horizontal curves, 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. It is defined as the highest continuous speed at which individual vehicle can travel with safety on the highway when weather conductions are conductive. It should be noted that 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.

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 satisfies nearly all drivers. For example, low design speed should not be selected for a rural collector road solely because the road is located in an area of flat topography, since motorists will tend to drive at higher speeds. In fact, Design speed depends on the functional classification of the highway, the topography of the area in which the highway is located, and the land use of the adjacent area. Table 1.1 shows suggested design speeds for different conditions.

Table 1.1: Design Speed

Type of Terrain	Metric					
	Design Speed (km/h) for Specified Design Volume (veh/day)					
	under 50	50 to 250	250 to 400	400 to 1500	1500 to 2000	2000 and over
Level	50	50	60	80	80	80
Rolling	30	50	50	60	60	60
Mountainous	30	30	30	50	50	50

1.2.4 Design vehicle

The diminutions, weight of the axle and operating characteristics of a vehicle influences the design aspects such as width of the pavement, radii of the curve, clearance, parking The vehicle type selected as the design vehicle is the largest that is likely to use the highway with considerable frequency. Figure 1.5 illustrates the design vehicle

Generally, vehicle which used roads are classified into category:

- Passenger cars: these include all passenger cars, including minivans, vans, pick-up trucks, and Sport vehicles.
- Trucks: these include all buses, single-unit trucks, combination trucks, and recreational vehicles

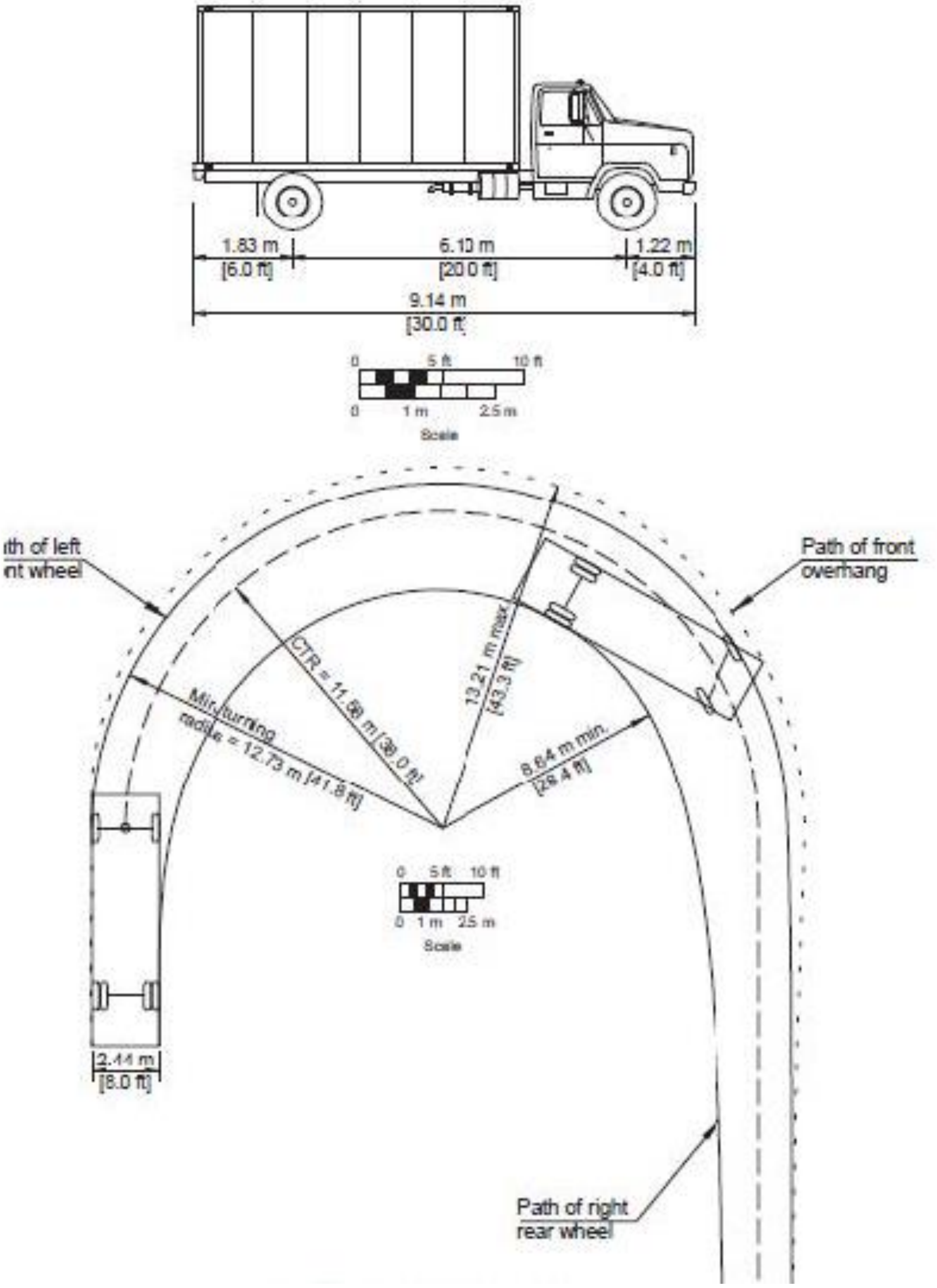


Figure 1.5: Design Vehicle

1.2.5 Cross-section of highway

Cross-section elements of a roadway include principal elements such as travel lanes, shoulders, and medians and marginal elements such as gutters, sidewalks, cross slopes, side slopes, back slopes, guard-rails. However, the availability of these elements depends of whether the road is in urban or rural areas. The element of cross-section will discussed later in detail. Figure 1.5 shows a typical cross-section of highway.

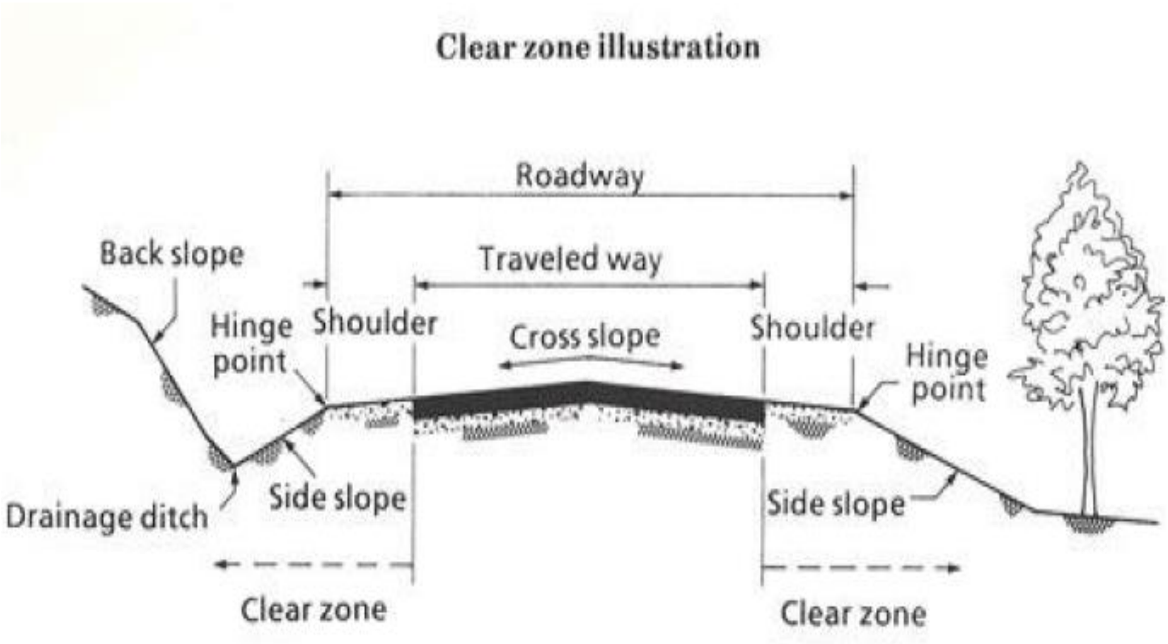


Figure 1.5: Typical cross-section of road

1.2.6 Presence of heavy vehicles on steep grades

The presence of heavy vehicles affects the required geometric design of road. This point is mostly related to next point.

1.2.7 Topography and environmental

For highway design, topography is generally classified into three groups:

1. Level terrain: this is relatively flat. Horizontal and vertical alignments are generally long or can be achieved without much construction difficulty or major expense. In addition, these horizontal and vertical alignments permitting heavy vehicles to maintain approximately the same speed as passenger cars. Grades are generally limited to 1 or 2 percent.

2. Rolling terrain: this type has natural slopes that often rise above and fall below the highway grade with occasional steep slopes that restrict the normal vertical and horizontal alignments. This terrain causing heavy vehicles to reduce their speeds substantially below those of passenger cars, but not to operate at crawl speeds.

3. Mountainous (hilly) terrain: it has sudden changes in ground elevation in both the longitudinal and transverse directions, thereby requiring frequent hillside excavations to achieve acceptable horizontal and vertical alignments. Furthermore, this type of terrain causing heavy vehicles to operate at crawl speed. Heavy vehicles are defined as any vehicle having a weight (Pounds) to horsepower ratio of 200 or greater. Crawl speed is defined as the maximum sustained speed that heavy vehicles can maintain on an extended upgrade

1.2.8 Level of service
1.2.9 Safety
1.2.10 Funds
1.2.11 Restrictions

1.3 Cross-section elements

As mentioned previously, the principal elements of a highway cross section consist of the travel lanes, shoulders, and medians (for some multilane highways). Marginal elements include and roadside barriers, kerbs, gutters, guard rails, sidewalks, and side slopes. Figure 1.6 shows a typical cross section for a two-lane highway, while Figure 1.7 shows that for a multilane highway. The features of the cross-section of the pavement influence the life of the pavement as well as the riding comfort and safety. Camber, kerbs, and geometry of various cross-sectional elements are important aspects to be considered in this regard. They are explained briefly in this lecture.

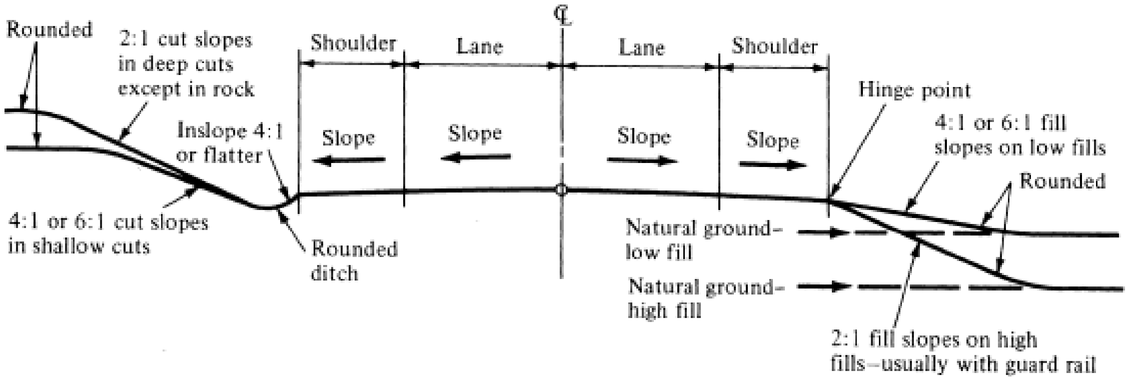


Figure 1.6: Typical cross-section for two-lane highway

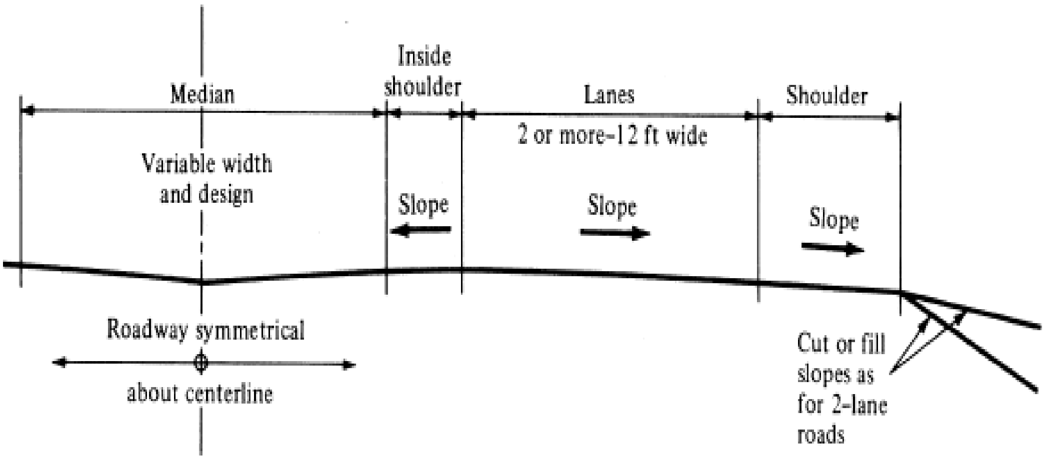


Figure 1.7: Typical cross-section for multilane highway

1.3.1 Pavement Surface Characteristics

For safe and comfortable driving four aspects of the pavement surface are important; the friction between the wheels and the pavement surface, smoothness of the road surface, the light refection characteristics of the top of pavement surface, and drainage to water

1.3.1.1 Friction

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

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

Various 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 tyre (new or old), and
- ✓ Speed and load of the vehicle.

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

1.3.1.2 Unevenness

It is always desirable to have an even surface, but it is seldom possible to have such a one. Even if a road is constructed with high quality pavers, it is possible to develop unevenness due to pavement failures. Unevenness affect the vehicle operating cost, speed, riding comfort, safety, fuel consumption and wear and tear of tyres. Unevenness index is a measure of unevenness which is the cumulative measure of vertical undulations of the pavement surface recorded per unit horizontal length of the road. An unevenness index value less than 1500 mm/km is considered as good, a value less than 2500 mm/km is satisfactory up to speed of 100 km/h and values greater than 3200 mm/km is considered as uncomfortable even for 55 km/h.

1.3.1.3 Light of reflection

- ✓ White roads have good visibility at night, but caused glare during daytime.
- ✓ Black roads has no glare during day, but has poor visibility at night
- ✓ Concrete roads has better visibility and less glare

It is necessary that the road surface should be visible at night and reflection of light is the factor that answers it.

1.3.1.4 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. ***Please Remember, the main enemy for pavement is water and water and water.***

1.3.2 Camber (Cross-slope)

Camber or cant is the cross slope provided to raise middle of the road surface in the transverse direction to drain off 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. Pavements on straight sections of two-lane and multilane highways without medians are slope from the middle downward to both sides of the highway, resulting in a transverse or cross slope, with a cross section shape that can be curved, plane or a combination of the two. A parabola is generally used for curved cross sections, and the highest point of the pavement (called the crown) is slightly rounded, with the cross slope increasing toward the pavement edge. Plane cross slopes consist of uniform slopes at both sides of the crown. Travelled-way cross slope should be adequate to provide proper drainage. Normally, cross slopes range from 1.5% to 2% for paved surfaces and 2% to 6% for unpaved surfaces. For unpaved surfaces, such as stabilized or loose gravel, and for stabilized earth surfaces, a 3% cross slope is desirable. Figure 1.8 shows different types of camber (cross-slope).

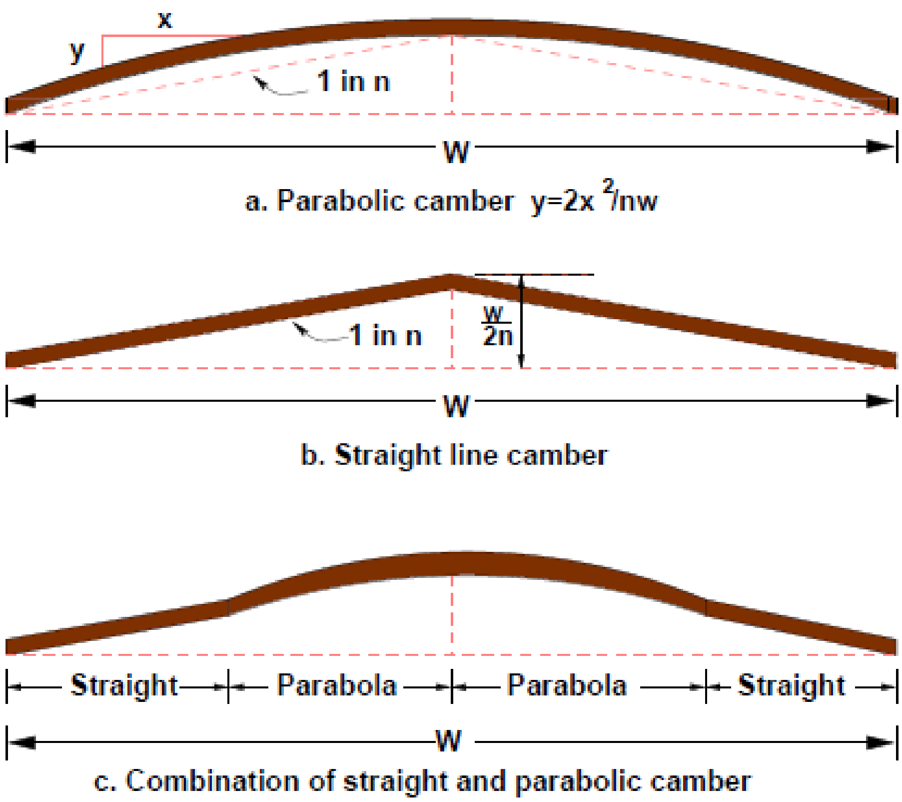


Figure 1.8: Different types of camber (cross-slope)

1.3.3 Width of Travel Lanes

Width of the carriageway or the width of the pavement depends on the width of the traffic lane and number of lanes. Width of a traffic lane (one lane width) depends on the width of the vehicle and the clearance. Side clearance improves operating speed and safety. In general, travel lane widths usually vary from 2.75m to 3.75m. Most arterials have 3.75 travel lanes since the extra cost for constructing 3.75m lanes over 3m lanes is usually offset by the lower maintenance cost for shoulders and pavement surface, resulting in a reduction of wheel concentrations at the pavement edges. On two lane, two-way rural roads, lane widths of 3m or 3.65m may be used, but two factors must be considered when selecting a lane width less than 3.65m wide. When pavement surfaces are less than 6.75m, the crash rates for large trucks tend to increase and, as the lane width is reduced from 3.65m, the capacity of a highway significantly decreases.

Lane widths of 3m are therefore used only on low-speed facilities. Lanes that are 3m wide are used occasionally in urban areas if traffic volume is low and there are extreme right-of-way constraints. It should be noted that the maximum permissible width of a vehicle is 2.44m 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. However, the side clearance required is about 0.53 m, on either side or 1.06 m in the centre. Therefore, a two-lane road require minimum of 3.5m for each lane.

In Iraq, a lane width of 3.75m is generally used for multilane highways while standard lane width is 3.60m. Table 1.2 represents the lane width as recommended by AASHTO Green book

1.3.4 Shoulders

The shoulder of a pavement cross section is always contiguous with the travelled lane to provide an area along the highway for vehicles to stop when necessary. Shoulder surfaces range in width from 0.6m on minor roads to 3.65m on major arterials. Shoulders are also used to laterally support the pavement structure. The shoulder width is known as either *graded* or *usable*, depending on the section of the shoulder being considered. The graded shoulder width is the whole width of the shoulder measured from the edge of the travelled pavement to the intersection of the shoulder slope and the plane of the side slope. The usable shoulder width is that part of the graded shoulder that can be used to accommodate parked vehicles. The usable width is the same as the graded width when the side slope is equal to or flatter than 4%. Minimum shoulder width of 1.80-2.40m may be considered for low- volume highways.

Asphalt and concrete – surfaced shoulders should be sloped from 2% to 6%, aggregate and untreated granular shoulders from 4% to 6%. In other words, slope of shoulder depends on the type of constructed materials.

Table 1.2 lane width as recommended by AASHTO

Metric					U.S. Customary				
Design Speed (km/h)	Minimum Width of Traveled Way (m) for Specified Design Volume (veh/day[a])				Design Speed (mph)	Minimum Width of Traveled Way (ft) for Specified Design Volume (veh/day[a])			
	under 400	400 to 1500	1500 to 2000	over 2000		under 400	400 to 1500	1500 to 2000	over 2000
30	6.0[b]	6.0	6.6	7.2	20	20[b]	20	22	24
40	6.0[b]	6.0	6.6	7.2	25	20[b]	20	22	24
50	6.0[b]	6.0	6.6	7.2	30	20[b]	20	22	24
60	6.0[b]	6.6	6.6	7.2	35	20[b]	22	22	24
70	6.0	6.6	6.6	7.2	40	20[b]	22	22	24
80	6.0	6.6	6.6	7.2	45	20	22	22	24
90	6.6	6.6	7.2	7.2	50	20	22	22	24
100	6.6	6.6	7.2	7.2	55	22	22	24	24
					60	22	22	24	24
					65	22	22	24	24
	Width of Shoulder on Each Side of Road (m)					Width of Shoulder on Each Side of Road (ft)			
All Speeds	0.6	1.5[c]	1.8	2.4	All Speeds	2.0	5.0[c]	6.0	8.0

[a] On roadways to be reconstructed, a 6.6-m [22-ft] traveled way may be retained where the alignment is satisfactory and there is no crash pattern suggesting the need for widening.

[b] A 5.4-m [18-ft] minimum width may be used for roadways with design volumes under 250 veh/day.

[c] Shoulder width may be reduced for design speeds greater than 50 km/h [30 mph] provided that a minimum roadway width of 9 m [30 ft] is maintained.

1.3.5 Medians

A median is the section of a divided highway that separates the lanes in opposing directions. The width of a median is the distance between the edges of the inside lanes, including the median shoulders. The functions of a median include:

- ✓ Providing a recovery area for out-of-control vehicles
- ✓ Separating opposing traffic
- ✓ Providing stopping areas during emergencies
- ✓ Providing storage areas for left-turning and U-turning vehicles
- ✓ Providing refuge for pedestrians
- ✓ Reducing the effect of headlight glare

Medians can either be raised, flush, or depressed as follows:

- Raised medians are frequently used in urban arterial streets because they facilitate the control of left-turn traffic at intersections by using part of the median width for left-turn-only lanes. Some disadvantages associated with raised medians include possible loss of control of the vehicle by the driver if the median is accidentally struck, and they cast a shadow from oncoming headlights, which results in drivers finding it difficult to see the curb.
- Flush medians are commonly used on urban arterials. They can also be used on freeways, but with a median barrier. To facilitate drainage of surface water, the flush median should be crowned. The practice in urban areas of converting flush medians into two-way left-turn lanes is common, since the capacity of the urban highway is increased while maintaining some features of a median.
- Depressed medians are generally used on freeways and are more effective in draining surface water. A side slope of 6% is suggested for depressed medians, although a slope of 4% may be adequate.

In general, median widths are in the range from 1.2m to 24m or even more at some cases. Median widths should be as wide as possible but should be balanced with other elements of the cross section and the cost involved. In general, the wider the median, the more effective it is in providing safe operating conditions and a recovery area for out-of-control vehicles. Figure 1.9 shows median cross-slope illustrations at different roads.

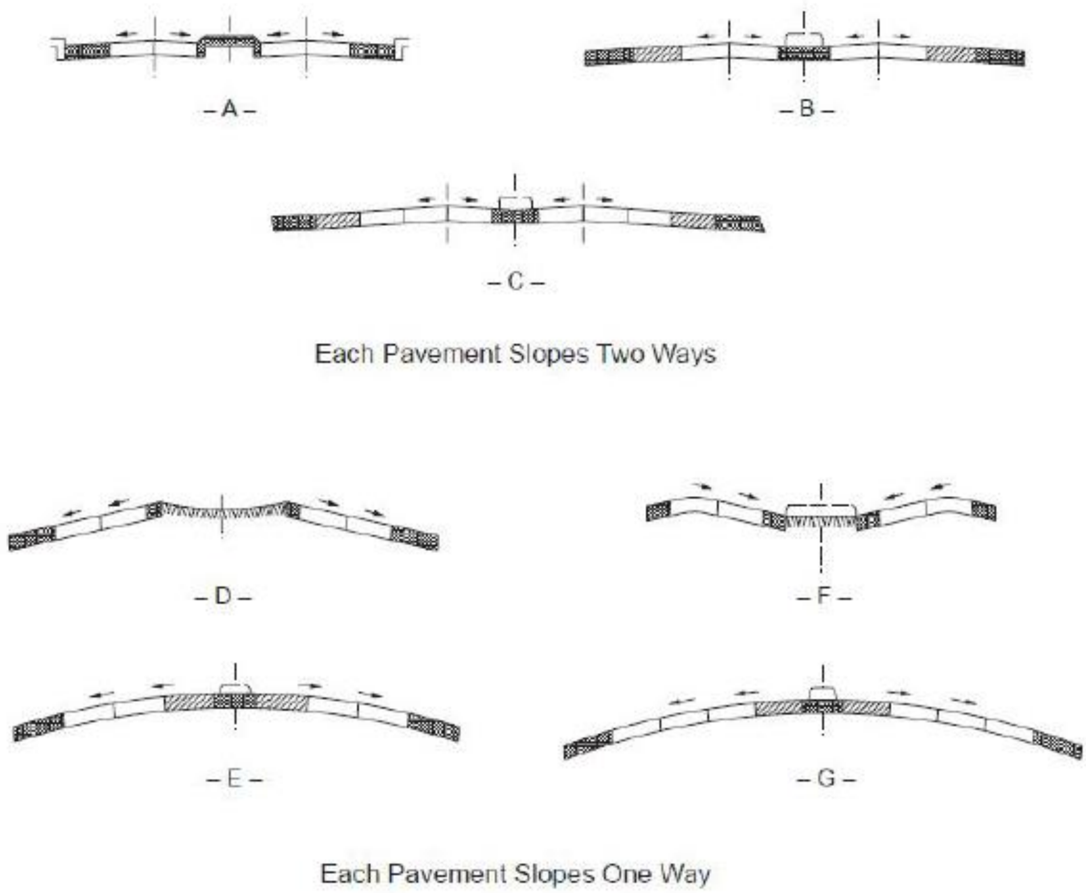


Figure 1.10: Median cross-slope illustrations at different roads

1.3.5 Kerbs

Kerbs are raised structures made of either Portland cement concrete or bituminous concrete (rolled asphalt kerb) that are used mainly on urban highways to delineate pavement edges and pedestrian walkways. kerb are also used to control drainage, improve aesthetics, and reduce right of way. They can be generally classified as either vertical or sloping Kerbs indicate the boundary between the carriage way and the shoulder or islands or footpaths.

Different types of kerbs are shown in Figure 1.11

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

- ✓ Semi-barrier kerbs: when the pedestrian traffic is high, these kerbs are provided. Their height is 15cm 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 kerbs: they are designed to discourage vehicles from leaving the pavement. They are provided when there is considerable amount of pedestrian traffic. They are generally placed at a height of 20cm 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.

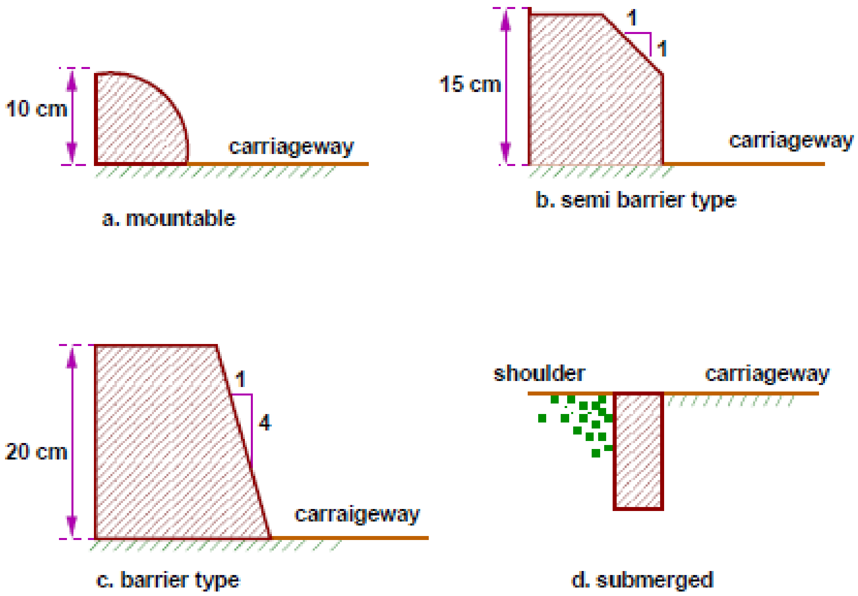


Figure 1.11: Different types of kerbs

1.3.5 Other elements

- **Sidewalks** are usually provided on roads in urban areas, but are uncommon in rural areas. Nevertheless, the provision of sidewalks in rural areas should be evaluated during the planning process to determine sections of the road where they are required. Sidewalks should have a minimum clear width of 1.25 m in residential areas and a range of 1.25 m to 2.5 m in commercial areas.
- **Cycle tracks** are provided in urban areas when the volume of cycle tracks is high Minimum width of 2 meter is required, which may be increased by 1 meter for every additional track.

1.4 Right -of -Way (ROW)

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. Sufficient right – of- way should be acquired in order to avoid the expense of purchasing developed property, with varying widths depending on local conditions. The right – of – way for a 2- lane highway in rural areas is recommended to have a minimum width of 30 m, with 37 m desirable. A minimum right-of-way width of 45m, and a desirable width of 76m are recommended for divided highways. Widths of 60 to 90 m have been used for divided highways without frontage roads. For Iraqi Expressway No One, a right- of- way width of 260 m has been provided, which included service roads.

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, 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.

1.5 Site Distance

In highway alignment design, the sight distance is a fundamental consideration that should be provided throughout the alignment. The safe and efficient operation of vehicles on the road depends very much on the visibility of the road ahead of the driver. Thus, the geometric design of the road should be done such that any obstruction on the road length could be visible to the driver from some distance ahead. This distance is called to be the 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:

- ✓ Stopping sight distance (SSD) or the absolute minimum sight distance
- ✓ Intermediate sight distance (ISD) is defined as twice SSD
- ✓ Overtaking sight distance (OSD) for safe overtaking operation
- ✓ Head light sight distance is the distance visible to a driver during night driving under the illumination of head lights
- ✓ Safe sight distance to enter into an intersection.

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. In practice, all these times are usually combined into a total perception-reaction time suitable for design purposes as well as for easy measurement. Many of the studies shows that drivers require about 1.5 to 2 secs under normal conditions. However, taking into consideration the variability of driver characteristics, a higher value is normally used in design. A reaction time of **2.5 sec** is considered adequate for design purposes.
- **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 at the moment the brakes are applied. However, practically, it is not possible to achieve 100% brake efficiency. Therefore, the sight distance required will be more when the efficiency of brakes are less. Also for safe geometric design, we assume that the vehicles have only 50% brake efficiency.
- **Frictional resistance between the tyre and the road**. The frictional resistance between the tyre and road plays an important role to bring the vehicle to stop. When the frictional resistance is more, the vehicles stop

immediately. No separate provision for brake efficiency is provided while computing the sight distance. This is taken into account along with the factor of longitudinal friction. It is has generally specified the value of longitudinal friction in between 0.35 to 0.4.

- **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. On the other hand, on descending a gradient, gravity also comes into action and more time will be required to stop the vehicle therefore, the requirement of Sight distance will be more in this case.

1.5.1 Stopping Sight Distance (SSD)

Stopping sight distance (SSD) is the minimum sight distance available on a highway at any spot having a sufficient length to enable the driver to stop a vehicle traveling at design speed, safely without collision with any other obstruction. In design consideration, the (site) safe stopping distance is one of the important measures in traffic engineering. It is the distance of vehicle travels from the point at which a situation is first perceived to the time of deceleration is complete. Drivers must have adequate time if they are to suddenly respond to a situation. The stopping sight distance is the sum of lag distance (or Perception-reaction distance) and braking distance as shown in Figure 1.12. These two components can be computed separately: Perception-reaction distance ($d_{reaction}$) travelled during perception-reaction time, and braking distance ($d_{braking}$) travelled after applying brakes.

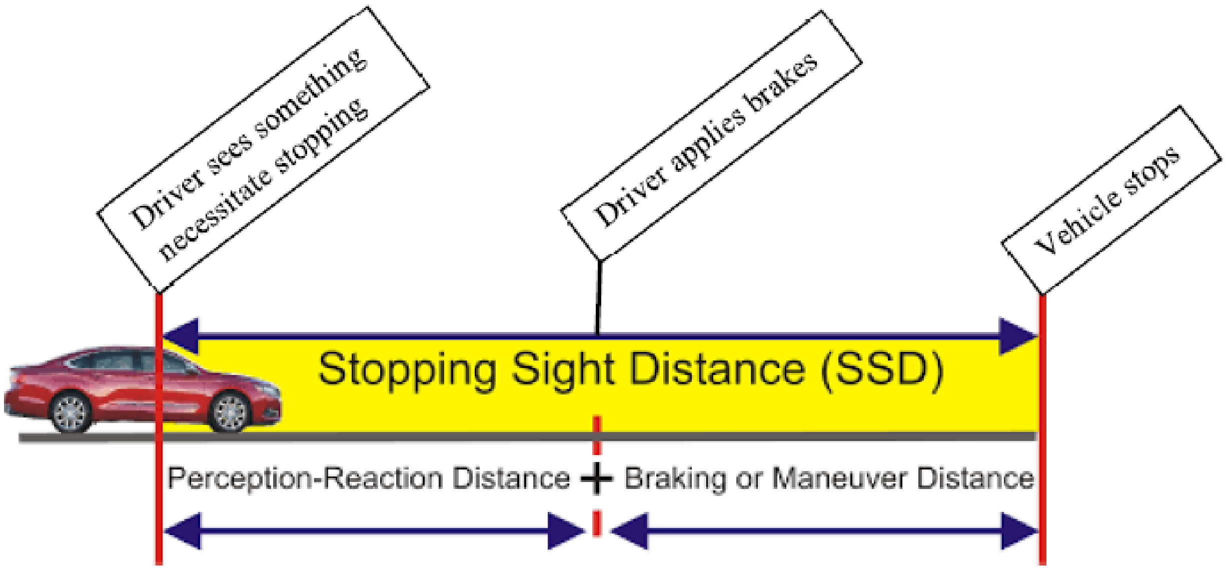


Figure 1.12: Diagrammatic representation of stopping sight distance components

- ***Perception-reaction distance***

As defined previously, a reaction time is the interval from the instant that the driver recognizes the potential hazard that need a stop until the instant that the driver actually applies the brakes. Vehicle speed and roadway environment probably also influence reaction time. Normally, a driver traveling at or near the design speed is more alert than one traveling at a lesser speed. A perception-reaction time of 2.5 sec. is considered adequate for design purposes. Perception-reaction distance in meters is calculated from the following equation.

$$\text{Reaction Distance} = v.t \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

Where: **v** is speed **(m/s)** and t is reaction time **(sec)**

$$\text{Reaction Distance} = 0.278\, V.t \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

Where: **V** is speed **(Km/h)** and t is reaction time **(sec)**

- ***Braking distance***

  Braking distance (D_B) in meters is computed from one of the following equations

$$D_B = 0.039\,\frac{v^2}{a} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \textbf{\textit{For flat terrain}}\ldots \quad (3)$$

$$D_B = \frac{v^2}{254\left[\left(\frac{a}{9.81}\right) \pm G\right]} \quad \ldots\ldots\ldots\ldots\ldots \textbf{\textit{For non-flat Terrain}}\ldots \quad (4)$$

Where V is speed in (Km/h), a is deceleration rate in (m/sec^2), G is grade of road in % and –ve and +ve signs should be used for downgrade and upgrade, respectively.

Therefore,

$$SSD = 0.278\ V.t + 0.039\,\frac{v^2}{a} \quad \ldots\ldots \textbf{\textit{For flat terrain}}\ldots\ldots\ldots \quad (5)$$

$$SSD = 0.278\ V.t + \frac{v^2}{254\left[\left(\frac{a}{9.81}\right) \pm G\right]} \quad . \textbf{\textit{For non-flat Terrain}}\ldots \quad (6)$$

1.5.2 Overtaking (Passing) Sight Distance (OSD or PSD)

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

It should be noted that passing sight distance only applies to two-lane, two-way highways because highways with additional lanes are not constrained by the risk posed by opposing traffic. The minimum passing sight distance for two-lane highways is determined as the sum of the following four distances as presented in Figure 1.13**:**

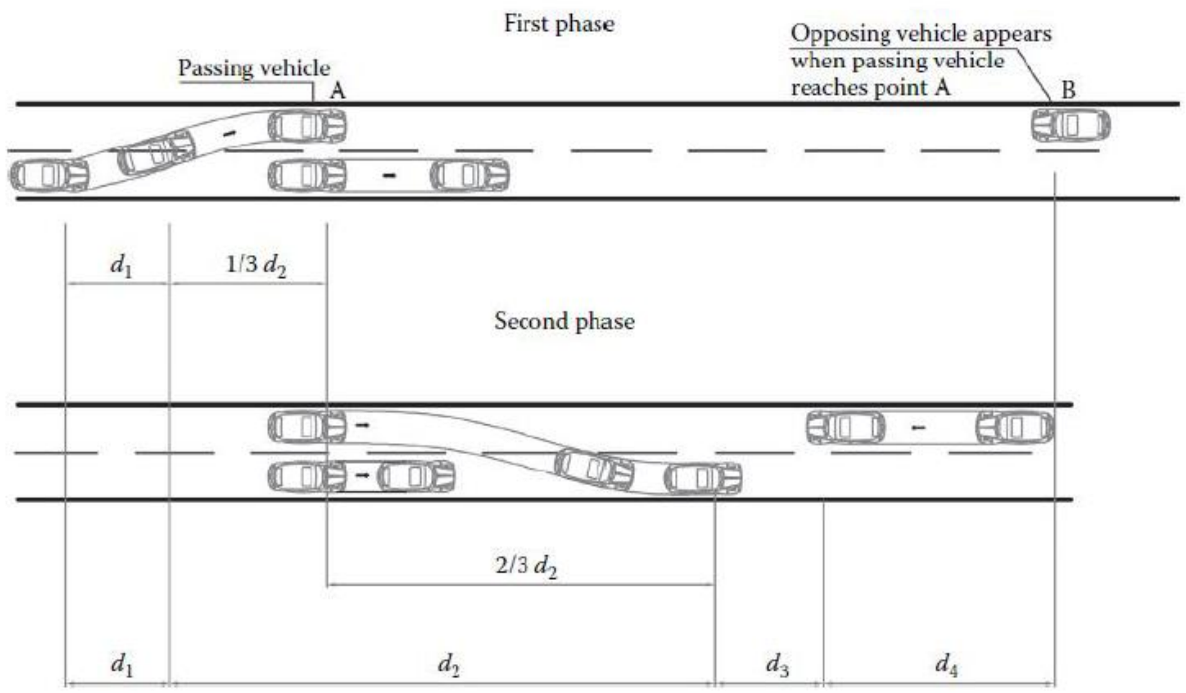


Figure 1.13: Passing sight distance elements

Therefore,

$$PSD = d_1 + d_2 + d_3 + d_4 \quad \text{.......} \quad (7)$$

Where d_1 is a distance traversed during perception-reaction time and during the initial acceleration to the point where the passing vehicle just enters the left lane.

$$\mathbf{d_1 = 0.278\ t_1\ [V - m + (a\ t_1 /2)]}$$

Where: t_1 is time of initial manoeuvre, s; a is average acceleration, km/h/s;) V is average speed of passing vehicle, km/h; m is difference in speed of passed vehicle and passing vehicle, taken as = 15 to 19 km/h.

d_2 is distance travelled during the time the passing vehicle is traveling in the left lane.

$$\mathbf{d_2 = 0.278\ V\ t_2}$$

Where: t_2 is time passing vehicle occupies the left lane, in sec (9.3s to 11.3s); and V is average speed of passing vehicle, in km/h.

d_3 = distance between the passing vehicle and the opposing vehicle at the end of the passing manoeuvre (such as, clearance distance). This distance at the end of the passing manoeuvre is assumed to be between 30 m to 75 m

d_4 is distance moved by the opposing vehicle during two thirds of the time the passing vehicle is in the left lane (usually taken to be 2/3 d_2). **$d_4 = 2/3 \times d_2$**

Table 1.3: PSD as recommended by AASHTO

Component of passing maneuver	Metric				US Customary			
	Speed range (km/h)				Speed range (mph)			
	50-65	66-80	81-95	96-110	30-40	40-50	50-60	60-70
	Average passing speed (km/h)				Average passing speed (mph)			
	56.2	70.0	84.5	99.8	34.9	43.8	52.6	62.0
Initial maneuver:								
a = average acceleration[a]	2.25	2.30	2.37	2.41	1.40	1.43	1.47	1.50
t_1 = time (sec)[a]	3.6	4.0	4.3	4.5	3.6	4.0	4.3	4.5
d_1 = distance traveled	45	66	89	113	145	216	289	366
Occupation of left lane:								
t_2 = time (sec)[a]	9.3	10.0	10.7	11.3	9.3	10.0	10.7	11.3
d_2 = distance traveled	145	195	251	314	477	643	827	1030
Clearance length:								
d_3 = distance traveled[a]	30	55	75	90	100	180	250	300
Opposing vehicle:								
d_4 = distance traveled	97	130	168	209	318	429	552	687
Total distance, $d_1 + d_2 + d_3 + d_4$	317	446	583	726	1040	1468	1918	2383

[a] For consistent speed relation, observed values adjusted slightly.

Note: In the metric portion of the table, speed values are in km/h, acceleration rates in km/h/s, and distances are in meters. In the U.S. customary portion of the table, speed values are in mph, acceleration rates in mph/sec, and distances are in feet.

1.5.2 Sight Distance at Intersection

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 Figure 1.14.

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.

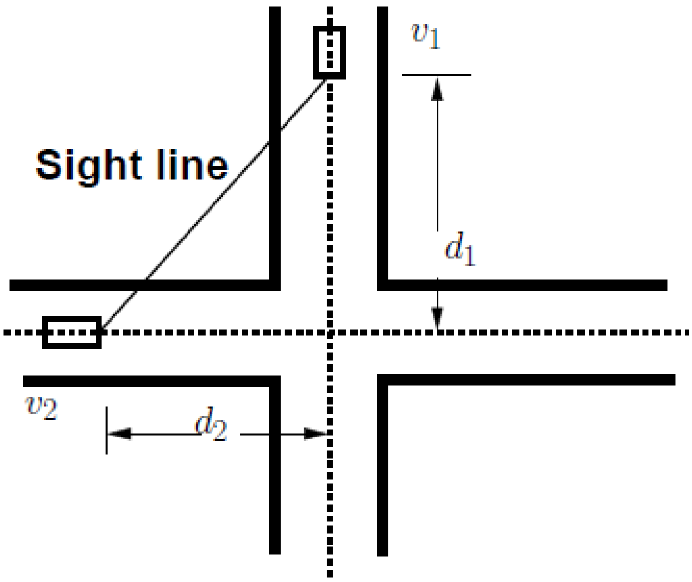


Figure 1.14: Sight distance at intersections

Example 2: For a Two-lane, Two-way (TLTW) highway, find: a- minimum sight distance to avoid head-on collision of two cars approaching at 90 km/h and 60 km/h? b- For the same conditions but the road has grade of downhill 3% (car of speed 90 km/h moves downward)? Use t =2.5 sec, a =3.5 m/sec^2

a)

For first car having speed of 90Km/h

SSD= $0.278*90*2.5 + \frac{90^2}{254\left[\left(\frac{3.5}{9.81}\right)\pm 0\right]} = 152\text{m}$

For first car having speed of 60Km/h

SSD= $0.278*60*2.5 + \frac{60^2}{254\left[\left(\frac{3.5}{9.81}\right)\pm 0\right]} = 81.5\text{m}$

Required total distance = 152+81.5 = <u>233.5m</u>

b)

For first car having speed of 90Km/h

SSD= $0.278*90*2.5 + \frac{90^2}{254\left[\left(\frac{3.5}{9.81}\right)-0.03\right]} = 160.14\text{m}$

For first car having speed of 60Km/h

SSD= $0.278*60*2.5 + \frac{60^2}{254\left[\left(\frac{3.5}{9.81}\right)-0.03\right]} = 85.1\text{m}$

Required total distance = 160.14+85.1 = <u>245.24m</u>

Example 3: A motorist traveling at 105 km/h on an expressway intends to leave the expressway using an exit ramp with a maximum speed of 55 km/h. At what point on the expressway should the motorist step on his brakes in order to reduce his speed to the maximum allowable on the ramp just before entering the ramp, if this section of the expressway has a downgrade of 3%? Use deceleration rate value as 3.4 m/sec^2

$$D_B = \frac{v1^2 - v2^2}{254\left[\left(\frac{a}{9.81}\right) \pm G\right]}$$

$$D_B = \frac{105^2 - 55^2}{254\left[\left(\frac{3.4}{9.81}\right) - 0.03\right]} = 99.5m$$

Example 3: Compute the safe passing sight distance of two lanes two-direction highway if the speed of passing vehicle was 85 km/ h and its acceleration was 0.65 m/s^2 and the clear distance between passing and opposing vehicles equal to 73meters and time of initial manoeuvre is 4 sec? use any standard values if needed?

PSD= $d_1 + d_2 + d_3 + d_4$

$d_1 = 0.278\ t_1\ [V - m + (a\ t_1 /2)]$

$d_1 = 0.278 \times 4\ [85 - 16 + (0.65 \times 3.6 \times 4/2)] = 81.93 = 82m$

$d_2 = 0.278 \times V \times t_2$ to find d_2 assume t_2 as 10 seconds

$d_2 = 0.278 \times 85 \times 10 = 236.3m$

d3 =73m

$d_4 = 2/3 \times d_2 = 2/3 \times 236.3 = 157.54m$

PSD=82 + 236.3 + 73 + 157.54=548.84=549m

Questions:

1- Find head light sight distance and intermediate sight distance for a vehicle having a speed of 65 Km/h (Hint: a=3.5 m/sec^2). Assume any standard value you would require.

2- Overtaking and overtaken vehicles are at 70 and 40 km/h respectively. Find PSD. (Hint: a=0.99 m/sec^2) Assume any standard value you would require.

1.6 Design of highway alignment

As explained previously, the design of highways necessitates the determination of specific design elements, which include the number of lanes, lane width, median type (if any) and width, length of acceleration and deceleration lanes for on- and off-ramps, need for truck climbing lanes and for steep grades, curve radii required for vehicle turning, and the alignment required to provide adequate stopping and passing sight distances. One of the most important element of those determinations is a design of alignment, which is mostly governed by the design speed of vehicle.

The alignment that follows the natural topography is the most economical one. However, it is not necessary to adopt the lowest cost alignment option since the designer always must adhere to design standards which might not exist on the natural topography. When they are designed, both vertical and horizontal alignments has to complete each other in order to ensure a consistent and safer roadway. In addition, horizontal and vertical alignments have to be well coordinated to avoid sudden changes and visibility problems.

The alignment of a highway is a three-dimensional problem measured in x, y and z coordinates as illustrated based on a driver's perspective and shown in Figure 1.15. However, in highway design practice, three-dimensional design computations are cumbersome, and, what is perhaps more important, the actual implementation and construction of design based on three-dimensional coordinates has historically been prohibitively difficult. As a consequence, the three-dimensional highway problem is reduced to two-dimensional alignment problems as illustrated in Figure 1.16.

One of the alignment problems in Figure 1.6 corresponds roughly to *x* and *z* coordinates and is referred to as horizontal alignment while y coordinates (elevation) and is refereed to vertical alignment

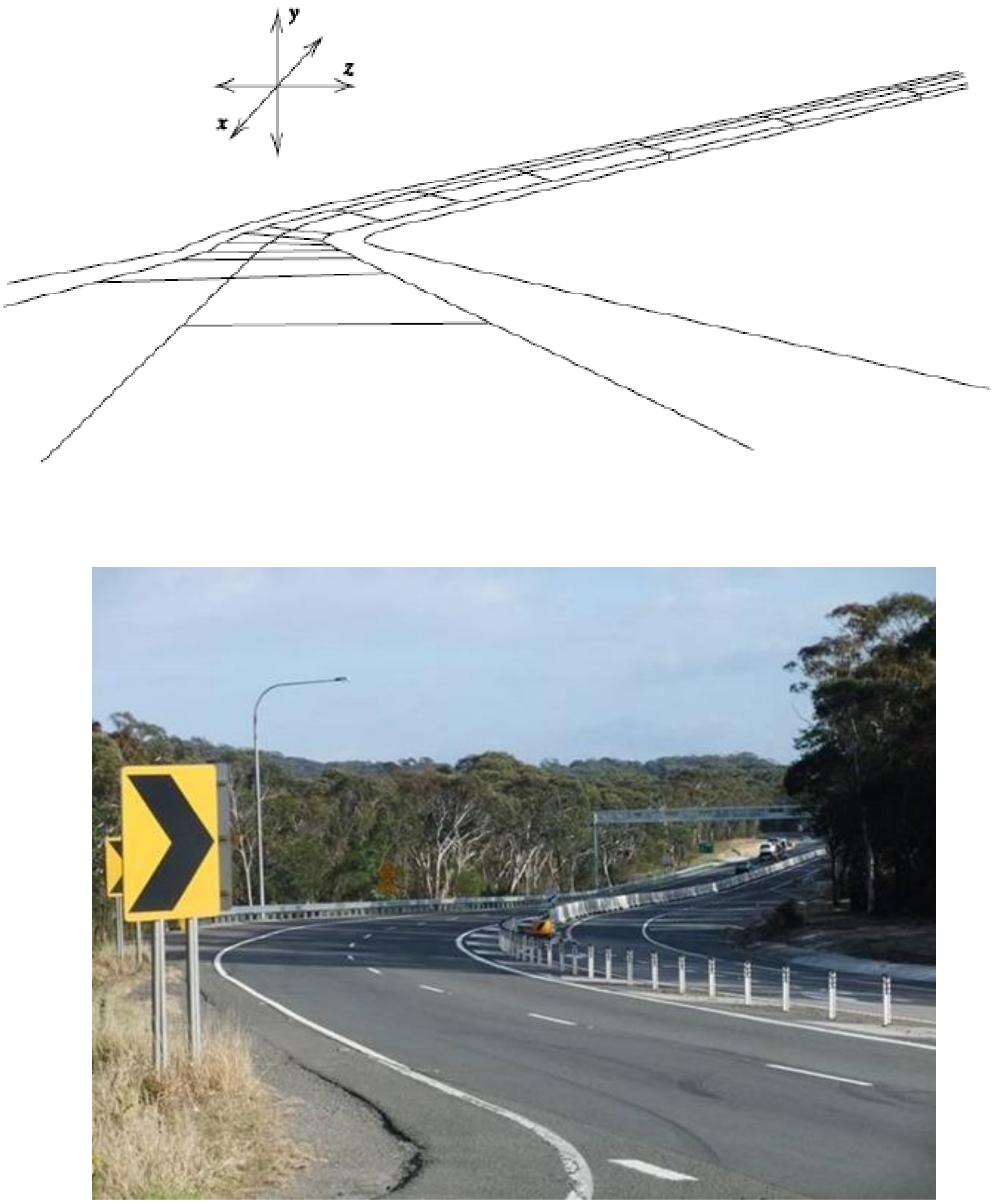


Figure 1.15: Highway alignment in three dimensions

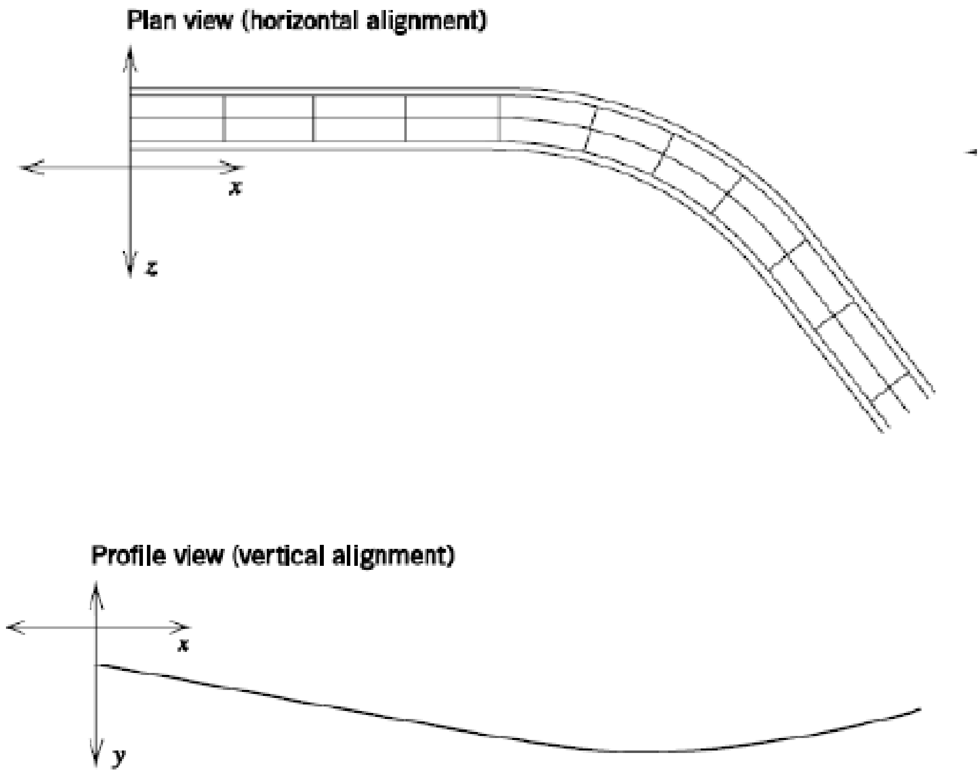


Figure 1.16: Highway alignment in two-dimensional views

1.6.1 Horizontal alignment

The critical aspect of horizontal alignment is the horizontal curve with focus on design of the directional transition of the roadway in a horizontal plane. In other words, a horizontal curve provides a transition between two straight (or tangent) sections of roadway as shown in Figure 1.17. These curves, which are circles segments, ensure smooth flow of traffic and "typically" same and consistent design speed of that provided on tangents. Therefore, a key concern in this directional transition is the ability of the vehicle to negotiate a horizontal curve However; in some design cases, it is difficult to ensure same design speed of tangents especially in urban area. This may consequently require a sharp radius. Furthermore, the design of highway is an interactive process where sometimes it is necessary to adjust the horizontal alignment based on the vertical alignment situation. Typical design of horizontal curve includes the determination of the minimum radius for a certain design speed and the other curve parameters those facilitating curve setting out.

a: Two-dimensional horizontal curve

b: three-dimensional horizontal curve

Figure 1.17: Horizontal curve

1.6.1.1 Tangents

Tangents is the straight parts of horizontal alignments, which could be expressed in terms of either bearings or azimuths. Azimuths represent angles turned clockwise from due north. On the other hand, bearings are expressed as angles turned either clockwise or counter clockwise from either north or south. For instance, the azimuth 290 is equivalent to the bearing north 70 west (or N70W) as presented in Figure 1.18. Generally, there are no limitations on the length of the tangents. In a flat terrain, the length of the tangents could be between 30 and 50 Km. It should be however noted that short curves at the end of long tangent must be avoided for safety consideration. Why?

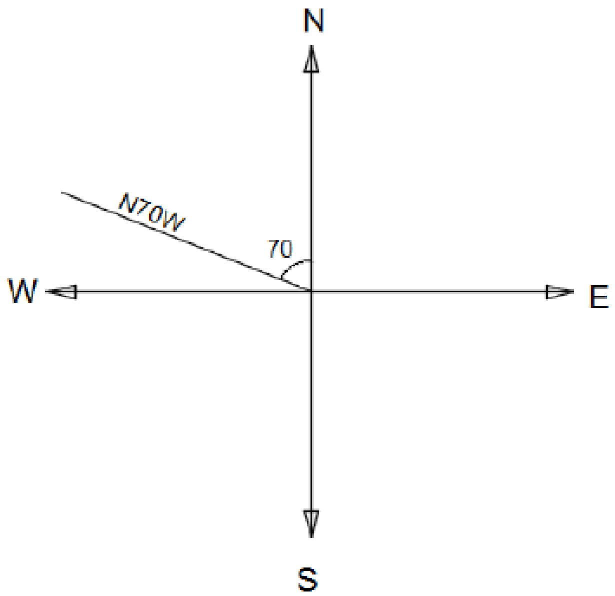


Figure 1.18: Horizontal tangent

1.6.1.2 Horizontal Curve

In connecting tangent (straight) sections of roadway with a horizontal curve, several options are available; simple circular curve, compound curves, reverse curves, and spiral curves. In general, from safety and comfort considerations, and to avoid shorter curve that create a kink impression, the minimum curve length should be (as a function of design speed in Kilometres):

L_{min}= 3V for major roads…………………………………………….....8

L_{min}= 6V for freeway roads……………………………………………9

In terms of maximum curve length, it should not exceed approximately 1 Km (1000 meters), with the preferred maximum length being 800 meters. On curves with very large radii, that is, greater than 3000 meters, the limitation on maximum curve length is no longer applicable.

Simple Curve: it is the most obvious type compared to others as shown in Figure 1.19. It is just a curve with a single, constant radius. It is widely used to maintain the design speed and provide the driver a safe and smooth transition from a tangent to other.

Reverse Curves: they generally consist of two consecutive curves that turn in opposite directions. Those curves are used to laterally shift the alignment of a highway. The type of these curves is generally circular with equal radii.

It should be however noted that reverse curves are not recommended because drivers may find it difficult to stay within their lane as result of sudden changes to the alignment. Figure 1.20 shows an example of reverse curves.

Compound Curves: they consist of two or more curves, usually circular, in succession. Those curves are used to fit horizontal curves to very specific alignment needs, such as interchange ramps, interaction curves, or difficult topography. Care should be taken in design process of such curves, as this will make it difficult for drivers to maintain their lane position as they transition from one curve to the next. Figure 1.21 presents an example of compound curves.

Spiral Curves: these curves are used with a continuously changing radius. In general, they are sometime used to transition a tangent section of roadway to a circular curve. In such a case, the radius of the spiral curve is equal to infinity where it connects to the tangent section and ends with the radius value of the connecting circular curve at the other end. Spiral curves are not often used but they are sometimes used on high-speed roadways with sharp horizontal curves and are sometimes also used to gradually introduce the superelevation of an upcoming horizontal curve. Figure 1.22 show an example of spiral curve.

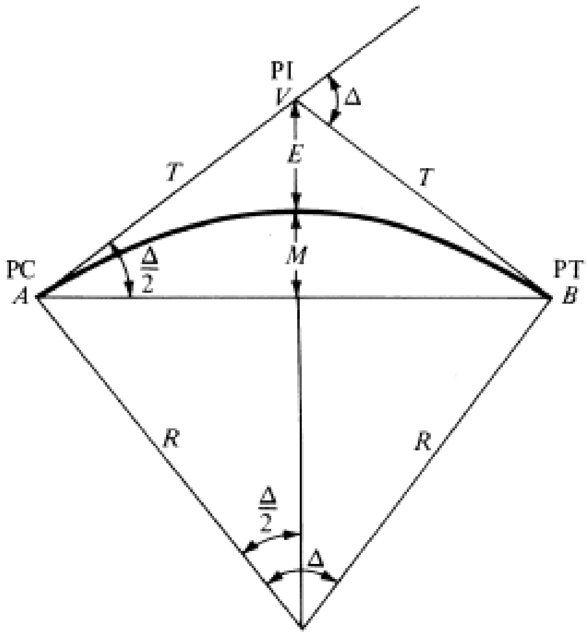


Figure 1.19: Horizontal curve

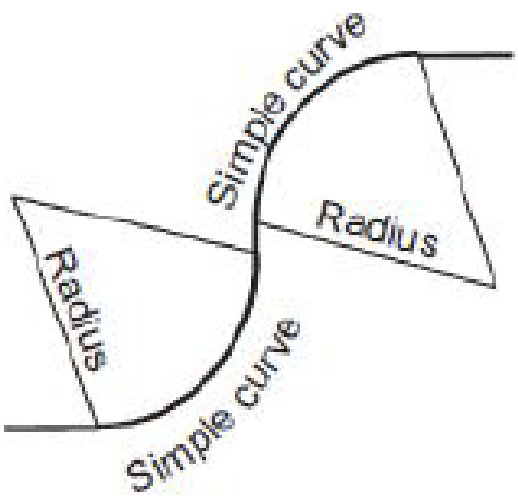


Figure 1.20: Reverse curve

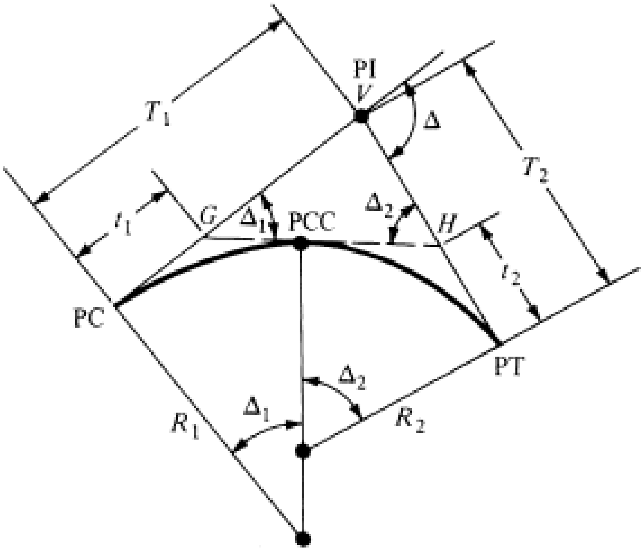


Figure 1.21: Compound curve

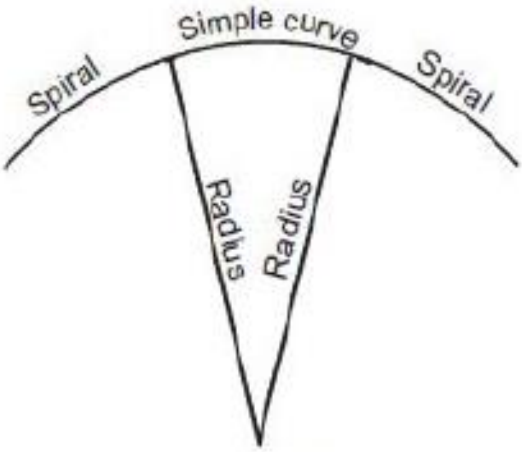


Figure 1.22: Spiral curve

1.6.2 Simple circular curve

Having mentioned that the simple circular curve is mostly and widely used in the design of highway alignment. The parameter of this kind of curves are previously presented in Figure 1.19. Those parameters can be calculated as follows:

$$D = \frac{3600}{2\pi R} = \frac{5729.58}{R} \quad \text{...} 10$$

$$L = \frac{2\pi R\Delta}{360°} = R\Delta_{rad} \quad \text{...} 11$$

$$T = R \tan\frac{\Delta}{2} \quad \text{...} 12$$

$$M = R\text{-}R \cos\left(\frac{\Delta}{2}\right) \quad \text{...} 13$$

$$E = \frac{R}{\cos\left(\frac{\Delta}{2}\right)} - R \quad \text{...} 14$$

$$C = 2R \sin\frac{\Delta}{2} \quad \text{...} 15$$

TC station = PI station – T ……………………………………………….16

CT station = TC station + L ……………………………………………….17

R= radius of curve, **L**= length of curve, **T**= tangent length/distance, **M**= middle ordinate, **Delta**= central angle (deflection angle), **D**= degree of curvature, **C**=chord length, **PI**=point of intersection, **TC**= tangent to curve point (or PC, point of curvature), and **CT** =curve to tangent point (or PT, point of tangency)

1.6.3 Superelevation

Superelevation 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 vehicles approaching a horizontal curve, there will be force resulted centripetal acceleration trying to push this vehicle outside the curve. This force is normally balanced by the force resulted from the friction between vehicles' tires and road surface. At high speeds and/ or low radius, the frictional force is not generally sufficient to balance the centrifugal force. For this reason, the carriageway should be super-elevated to increase the resistance as shown below. In order to find out how much this raising should be, the following analysis may be done as presented in Figure 1.24 whereas; Figure 1.23 shows a general example of superelevation.

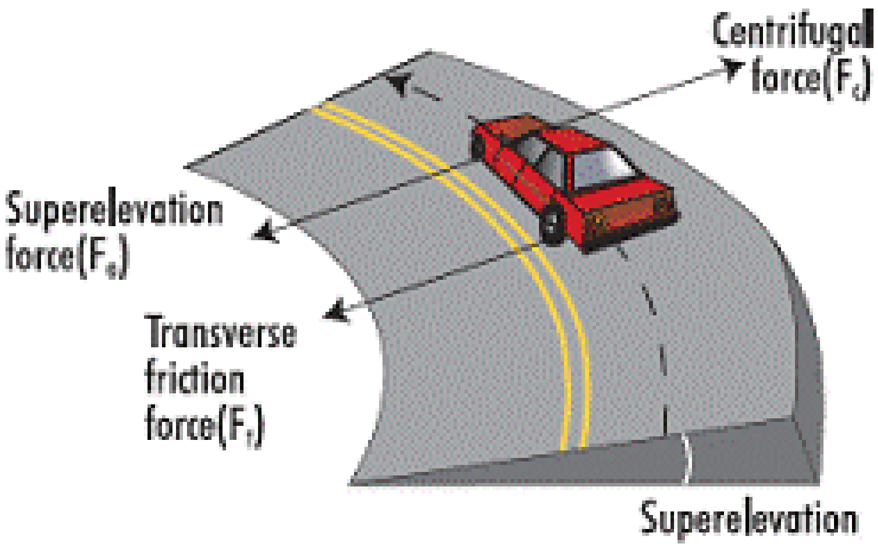


Figure 1.23: Superelevation

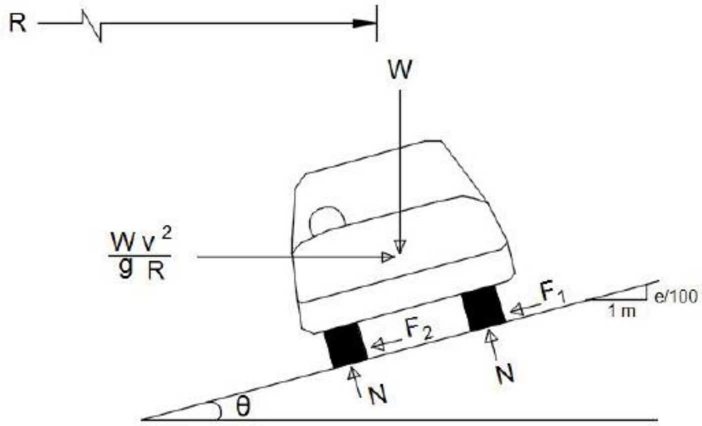


Figure 1.24: Vehicle on curves, acting forces

$\sum Fy = 0$

N= P sinθ + W cos θ *hint* P sin $\theta \cong$ zero

N= W cos θ 18

$\sum Fy = 0$

W sin θ + N. f = P cos θ...19

W sin θ + w cos θ. f = P cos θ 20

$$P = \frac{wv^2}{gR} \quad \text{...21}$$

By dividing Eq. 20 by w. cos θ, we get

$$\tan\theta + f = \frac{v^2}{gR}, \quad \text{Hint: } \tan\theta = e = \text{superelevation}$$

$$e + f = \frac{v^2}{gR} \quad (v \text{ in m/sec}) \text{ ....22}$$

$$e + f = \frac{V^2}{127\,R} \quad (V \text{ in Km/hr}) \text{ ....23}$$

where,

e = superelevation, f = coefficient of friction (side friction), V= Design Speed and R = Radius.

1.6.3.1 Minimum radius of curvature

In design consideration, when a minimum radius of curve is applied, the superelvation and coefficient of fraction have to be at maximum values. Therefore, the equation that governs the minimum radius of curvature is calculated based on the following equation;

$$R_{min} = \frac{V^2}{127(e_{max}+f_{max})} \qquad \ldots\ldots 24$$

The maximum value for the rate of superelevation is affected by several factors such as:

1- Location of the highway (that is, whether it is in an urban or rural area),
2- Weather conditions (such as the occurrence of snow),
3- Distribution of slow moving traffic within the traffic stream.

In general, for highways located in rural areas where is no snow or ice, a maximum superelevation rate of 0.10 generally is used. For highways located in areas with snow and ice, values ranging from 0.08 to 0.10 are used; while for expressways in urban areas, a maximum superelevation rate of 0.08 is used. Values of 4% or 6% for e_{max} are preferred options for urban streets, as those roads are usually not superelevated because relatively low speeds on local urban roads are applied. Figure 1.25 shows a relation between side friction and design speed.

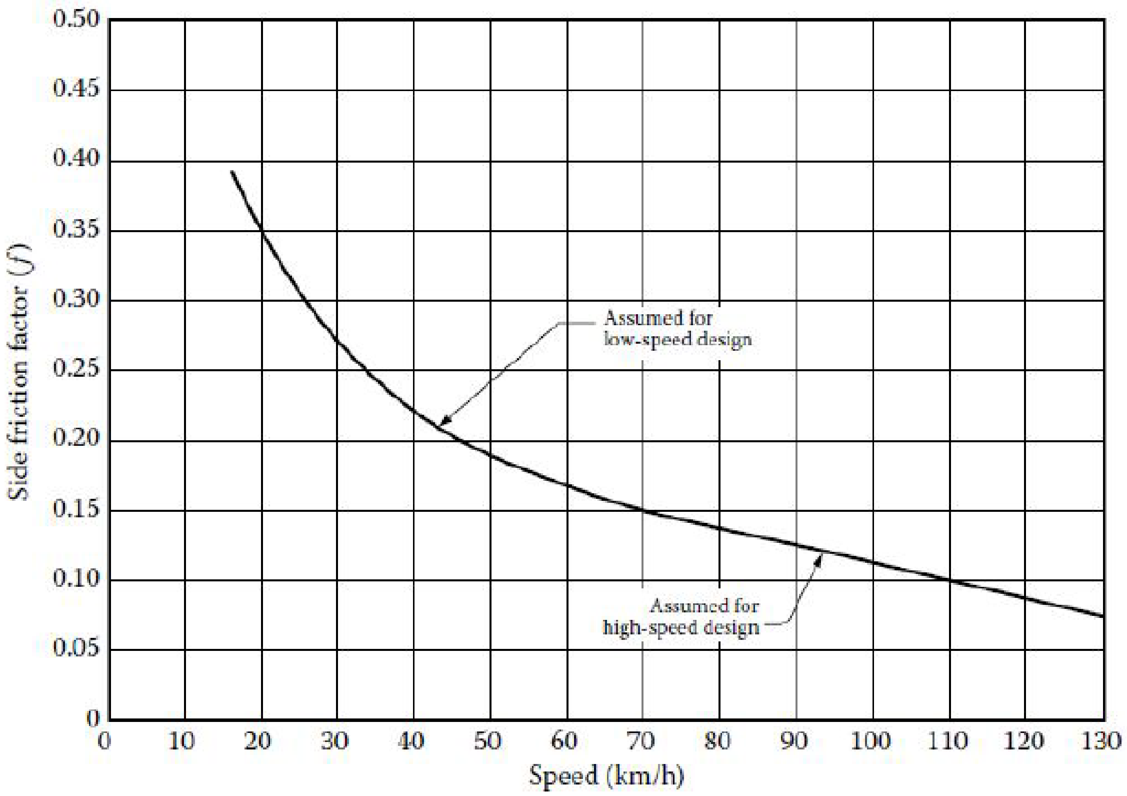


Figure 1.25: Recommended maximum side friction factors for different design speeds

1.6.3.2 Design superelevation rate

Indian Road Congress (IRC)

Indian road congress has formulated the a procedure to compute superelevation rate and minimum radius of horizontal curve as follows:

Step One: Find e for 75% design speed and neglecting the effect of coefficient of frication:

$$e_1 = \frac{(0.75v)^2}{127\ R} \quad \text{...} 25$$

Step Two: If $e_1 \leq$ maximum superelvation (in Iraq emax = 0.08), then

$e = e_1 = \dfrac{(0.75v)^2}{127\ R}$ if elase, $e_1 > e_{max}$ then go next step

Step Three: Find f for design speed and e_{max}

$$f = \frac{v^2}{127\,R} - e \quad \text{...26}$$

if $f < f_{\max}$ then $e = e_{max}$ is safe for the design speed, otherwise go to Step Four.

Step Four: Find the allowable speed V_a for $e = e_{max}$ and $f = f_{max}$

$$R = \frac{V^2}{127(e+f)} \quad \text{...27}$$

If Va > V then the design is adequate otherwise apply speed control measures.

AASHTO Procedure

AASHTO's geometric design policy has developed charts for several superelevation (e_{max}) in both metric and English units. See the attached charts with handout

Questions

Q1*: The point of intersection (P.I.) of two tangents is at station 15+20. The radius of curvature is 275m deflection angle is 520. Find the length of the curve, the station for the TC (or PC) and TC (orPT), and all other relevant characteristics of the curve (i.e., C., M, and E).*

Q2*: A horizontal curve is designed with a 725m radius. The curve has a tangent lengthy 140m and PI is at station 3 + 103. Determine the stationing of the PT.*

Q3: *A national highway passing through a rolling terrain has two horizontal curves of 450 m and 150 m. Design the required superelevation for the curves which are applicable to accommodate speed of 80 km/h. and f value is 0.15? Use the IRC guidelines. Adopt* e_{max} *=0.07*

Q4: A highway in urban area has a design speed of 80Km/h and a maximum superelevation rate of 8%. Design a suitable horizontal curve. Use *f*= 0.14, road camber 2%.

Q5: *Solve the previous example using AASHTO procedure*

1.6.3.3 Attainment of superelevation

It is essential that the change from a crowned cross section to a superelevated one be achieved without causing any discomfort to motorists or creating unsafe conditions. One of three methods can be used to achieve this change on undivided highways.

1. A crowned pavement is rotated about the profile of the centreline.

2. A crowned pavement is rotated about the profile of the inside edge.

3. A crowned pavement is rotated about the profile of the outside edge.

Figures 1.26 and 1.27 is a schematic of Method 1. This is the most commonly used method since the distortion obtained is less than that obtained with other methods. The procedure used is first to raise the outside edge of the pavement relative to the centreline, until the outer half of the cross section is horizontal. The outer edge is then raised by an additional amount to obtain a straight cross section. Note that the inside edge is still at its original elevation. The whole cross section is then rotated as a unit about the centreline profile until the full superelevation is achieved.

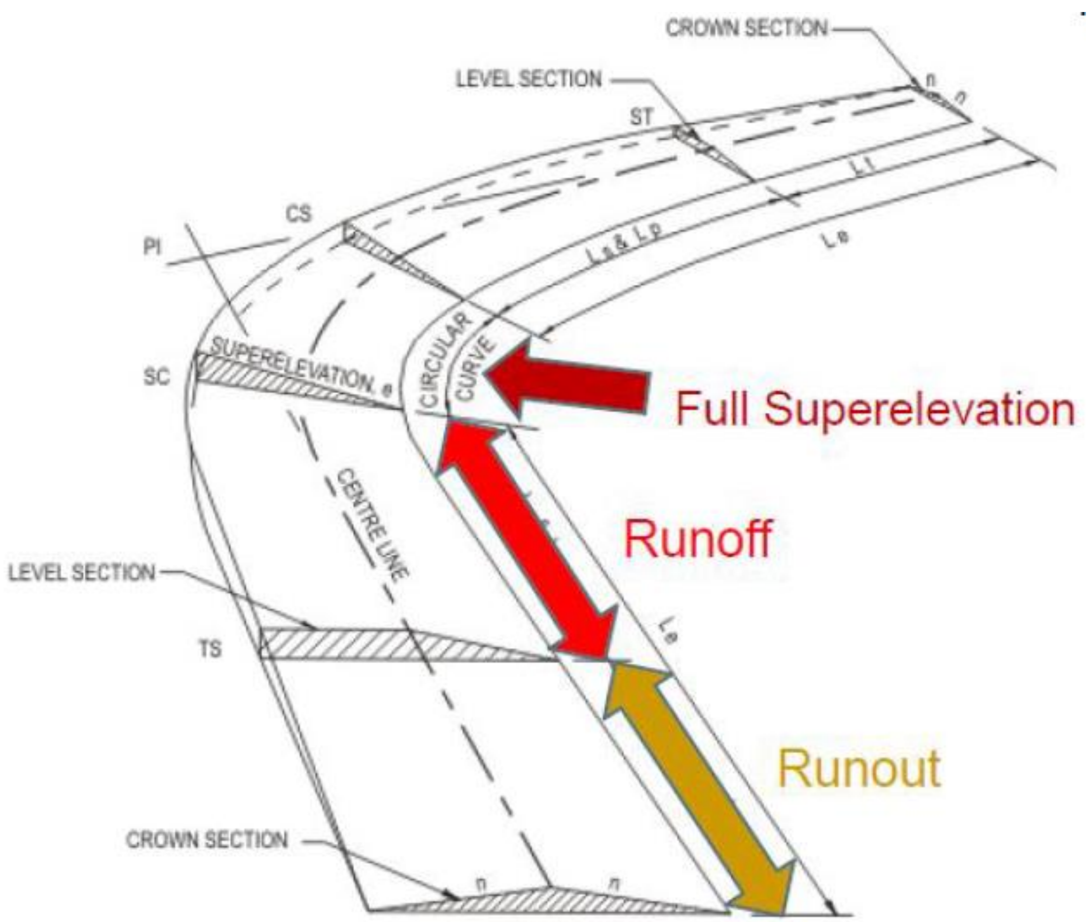


Figure 1.26: Attainment of superelevation

Figure 1.28 illustrates Method 2 where the centreline profile is raised with respect to the inside pavement edge to obtain half the required change, while the remaining half is achieved by raising the outside pavement edge with respect to the profile of the centreline. Note that the inside edge and centreline are still at their original elevations. The whole cross section is then rotated as a unit about the inside edge point until the full superelevation is achieved (the elevation of inside edge, remains constant from the beginning to the ending of rotation process).

Method 3, demonstrated by Figure 1.29, is similar to Method 2 with the only difference being a change affected below the outside edge profile.

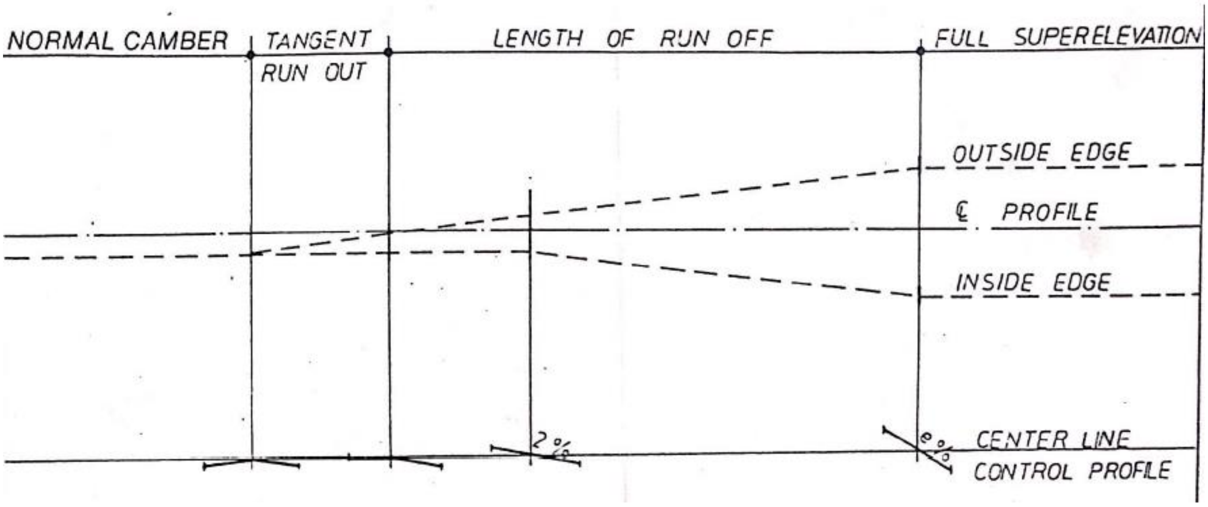


Figure 1.27: A crowned pavement is rotated about the profile of the centreline

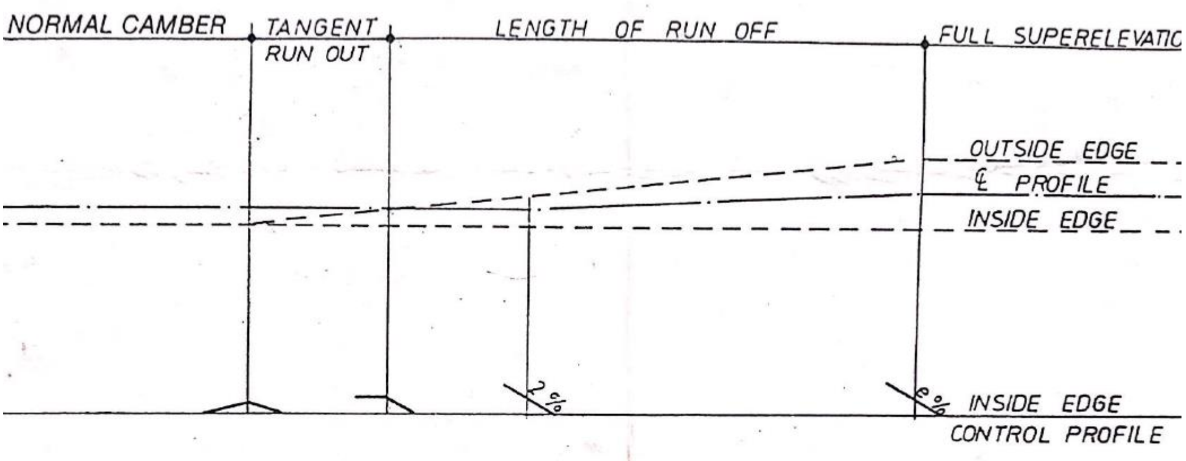


Figure 1.28: A crowned pavement is rotated about the profile of the outside edge

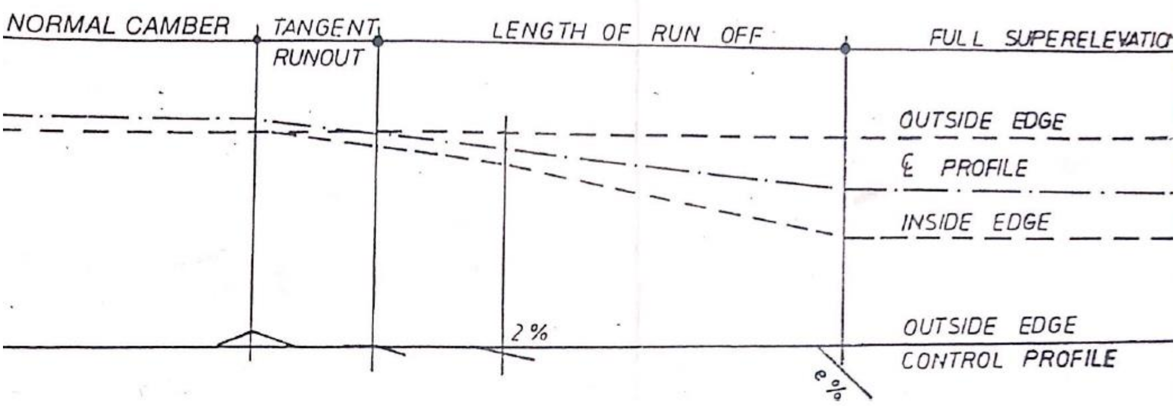


Figure 1.28: A crowned pavement is rotated about the profile of the inside edge

Tangent Runout Length

Length of tangent roadway needed to accomplish a change in outside-lane cross slope from normal cross slope rate to zero (flat).

Length of superelevation Runoff when spiral curves are not used

Superelevation is uniformly applied to provide a smooth transition from a normal crown section to a full superelevation section as shown in Figure 1.26. Two-thirds of superelevation runoff occurs on the tangent segment prior to the PC and then again, after the PT. One-third of the superelevation runoff occurs on the curve between the PC and the PT at each end of the curve. The rest of the curve is in a full superelevation section. The crown runoff that transitions from a normal crown to a flat crown (and vice versa) is placed outside each superelevation runoff section.

Minimum superelevation runoff length

It is the length required to change the cross-section from adverse crown removed to the full superelevated cross section. It can be estimated from the following formula

$$L_r = \left(\frac{3.6\,e}{G}\right).\,a \qquad \text{.....28}$$

Where: Lr is superelevation runoff length, e is full superelevation (%), G is relevant gradient (%) as presented in Table 1.4, a is multilane adjustment factor (dimensionless) as shown in Table 1.5

Table 1.4: Recommended relative gradient values

Metric			US Customary		
Design speed (km/h)	Maximum relative gradient (%)	Equivalent maximum relative slope	Design speed (mph)	Maximum relative gradient (%)	Equivalent maximum relative slope
20	0.80	1:125	15	0.78	1:128
30	0.75	1:133	20	0.74	1:135
40	0.70	1:143	25	0.70	1:143
50	0.65	1:150	30	0.66	1:152
60	0.60	1:167	35	0.62	1:161
70	0.55	1:182	40	0.58	1:172
80	0.50	1:200	45	0.54	1:185
90	0.47	1:213	50	0.50	1:200
100	0.44	1:227	55	0.47	1:213
110	0.41	1:244	60	0.45	1:222
120	0.38	1:263	65	0.43	1:233
130	0.35	1:286	70	0.40	1:250
			75	0.38	1:263
			80	0.35	1:286

Table 1.5: Recommended adjustment factors

Roadway Type	α
two-lane undivided highway (w = 3.6 m)	1
four-lane divided highway (w = 7.2 m)	1.5
standard ramp (w = 4.8 m)	1.167
standard loop (w = 5.5 m)	1.264

Minimum superelevation runoff length (AASHTO 2011)

The minimum superelevation runoff length is computed on the basis of comfort and aesthetic purposes according to AASTO's geometric design policy using the following formula. It should be however noted that the length calculated on the basis of the following equation represent the minimum value and it is desirable to use more length especially for high type alignments.

$$L_r = \frac{wn_1 * e_d * b_w}{\Delta} \qquad \text{...29}$$

Where:

Lr is the minimum length of superelevation runoff, m

w is width of one traffic lane, m

n_1 is the number of lanes rotated.

e_d is the design superelevation rate, %

b_w is the adgusement factor for number of lanes rotated

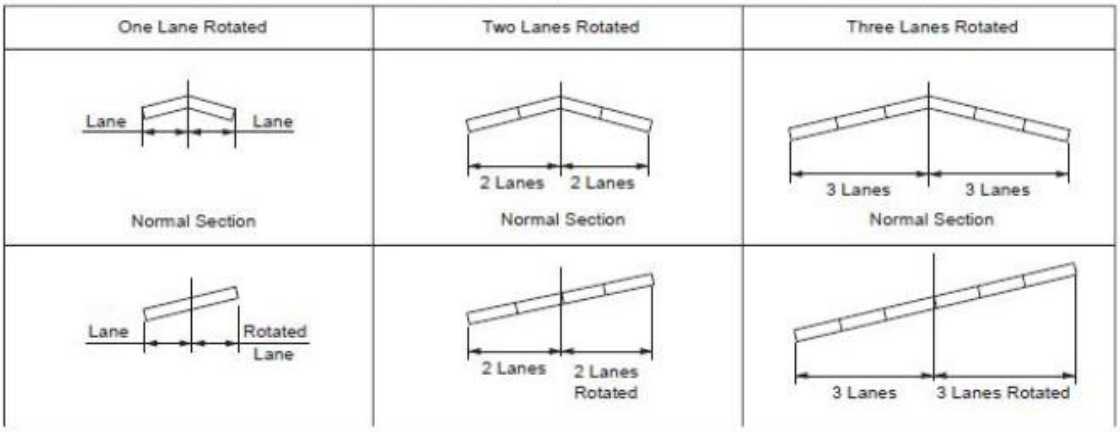
Δ is the mixumum relative gradient, %

Table 1.6: Maximum relative gradient (AASHTO method)

Metric		
Design Speed (km/h)	Maximum Relative Gradient (%)	Equivalent Maximum Relative Slope
20	0.80	1:125
30	0.75	1:133
40	0.70	1:143
50	0.65	1:154
60	0.60	1:167
70	0.55	1:182
80	0.50	1:200
90	0.47	1:213
100	0.44	1:227
110	0.41	1:244
120	0.38	1:263
130	0.35	1:286

Table 1.7: Adjustment factor for numbers of lanes rotated

Metric			U.S. Customary		
Number of Lanes Rotated, n_1	Adjustment Factor,* b_w	Length Increase Relative to One-Lane Rotated, $(= n_1 b_w)$	Number of Lanes Rotated, n_1	Adjustment Factor,* b_w	Length Increase Relative to One-Lane Rotated, $(= n_1 b_w)$
1	1.00	1.0	1	1.00	1.0
1.5	0.83	1.25	1.5	0.83	1.25
2	0.75	1.5	2	0.75	1.5
2.5	0.70	1.75	2.5	0.70	1.75
3	0.67	2.0	3	0.67	2.0
3.5	0.64	2.25	3.5	0.64	2.25



* $b_w = [1 + 0.5\,(n_1 - 1)]/n_1$

Length of superelevation Runoff when spiral curves are used

When spiral curve is used, the length of transition curve is equal to the length of Runoff

Minimum superelevation runout length (AASHTO 2011)

Two factors govern the tangent runout length, adverse cross slope value which is intended to be removed and the rate at which it is removed. Smooth pavement edge can be achieved by making the rate of cross slope removal equal to the relative gradient of superelevation runoff. Based on this concept, minimum length of tangent runout can be computed by the following equation

$$L_t = \frac{e_{NC}}{e_d} \ L_r \quad \ldots\ldots 30$$

Where, L_t is minimum length of tangent runout, *m*; e_{NC} is normal corse slpoe rate, *%*; e_d is design superelevation rate, *%* and L_r is the minimum length of superelevation runoff.

However, in case of using spiral curve the following equation is used to determine the minimum tangent runout length:

$$L_t = \frac{e_{NC}}{e_d} \ L_s \quad \ldots\ldots 31$$

Where, L_s is length of spiral curve.

1.6.4 Spiral (Transition) curve

A spiral curve is a geometric feature that can be integrated on to a regular circular curve. The spiral provides a gradual transition from moving in a straight line to moving in a curve around a point (or vise-verse). In other words, the use of transition curves provides a vehicle path that gradually increases or decreases the radial force as the vehicle enters or leaves a circular curve. Furthermore, spiral curves can be also used to introduce superelevation transition and used for aesthetic purposes especially the high type roadways.

Length of Spiral Curves

If the transition curve is a spiral, the degree of curve between the tangent and the circular curve varies from zero at the tangent end to the degree of the circular curve (R=Rc) at the curve start. However, when the transition is placed between two circular curves, the degree of curve varies from that of the first circular curve to that of the second circular curve.

Table 1.8: Maximum radius use in spiral transition curves

Metric	
Design speed (km/h)	Maximum radius (m)
20	24
30	54
40	95
50	148
60	213
70	290
80	379
90	480
100	592
110	716
120	852
130	1000

The following equations 32 and 33 are used by some highway agencies to compute the minimum length of a spiral transition curve. It should be noted that the minimum length should be the larger of the values obtained from equation 32 and 33.

$$L_{s,min} = \frac{V^3}{46.7RC} \quad \text{...32}$$

$$L_{s,min} = \sqrt[2]{24RP_{min}} \quad \text{..33}$$

Where; $L_{s,min}$ is the minimum length of spiral curve (m) ; V is speed (Km/h); R is the radius of curve (m); C is the rate of increase of radial acceleration $m/s^2/s$ Values range from 0.3 to 0.9 $m/s^2/s$ (1 to 3 ft/s^3) have been used for highways; P_{min} is the minimum lateral offset between the tangent and the circular curve, **0.2m**.

$$L_{s,max} = \sqrt[2]{24RP_{max}} \quad \text{..34}$$

Where;

$L_{s,max}$ is the maximum length of spiral curve (m)

P_{min} is the maximum lateral offset between the tangent and the circular curve, **1m**

<u>1.6.5 Stopping sight distance and horizontal curve</u>

Adequate stopping sight distance must be provided in the design of horizontal curves. Sight distance restrictions on horizontal curves occur when obstructions are present, as presented in Figure 1.29. Such obstructions are frequently encountered in highway design due to the cost of right-of-way acquisition or the cost of moving earthen materials, such as rock outcroppings. When such an obstructions exists, the stopping sight distance is measured along the horizontal curve from the cntre of travelled lane (the assumed location of the driver's eyes)

As shown in Figure 1.29, for a specified stopping distance, some distance M (the middle vordinate of a curve that has an arc length equal to the stopping sight distance) must be visually cleared so that the line of sight is such that sufficient stopping sight distance is available.

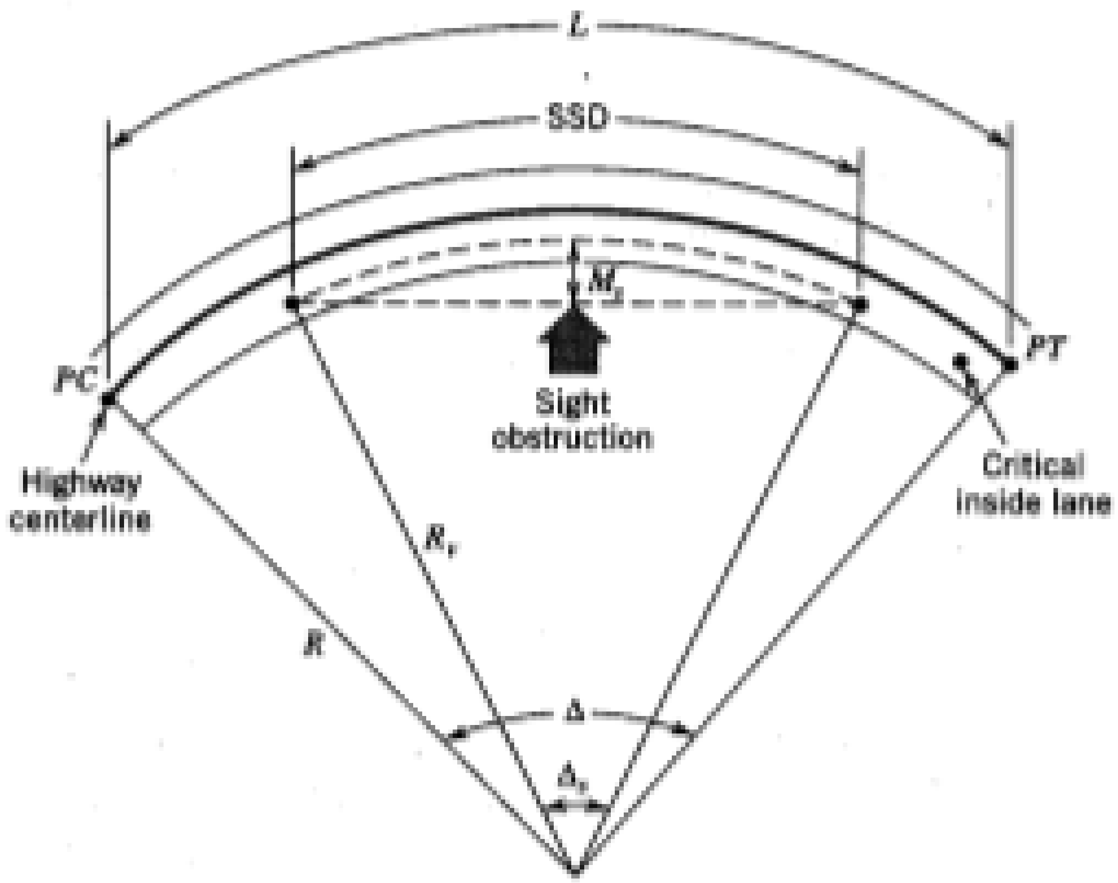


Figure 1.29: Illustration of distance to obstruction

The required distance to obstruction (m) necessary to provide a stopping sight distance (SSD) could be computed by the following formula:

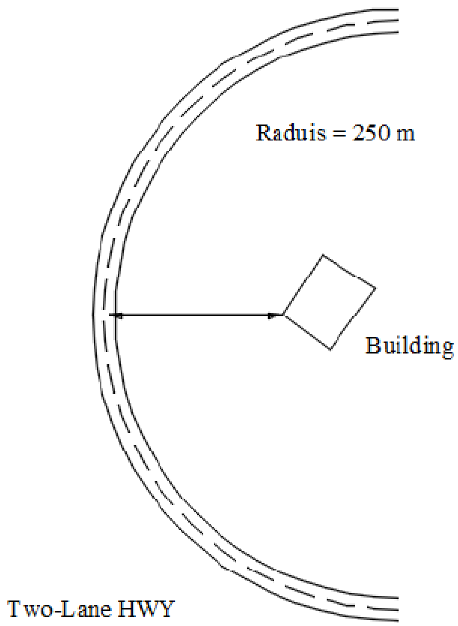
$$m = R\left(1- \cos \frac{28.65\,SSD}{R}\right) \quad \text{.........35}$$

m= distance to obstruction, m.

R= radius of curve, m.

SSD= sight distance, m.

Example 5: A new service building needs to be constructed near the centre of curve as shown in figure below. Compute the distance from the centre of the inside lane beyond which the building can be constructed so that sight distance on the curve will not be affected. The design speed of the existing road is 60 km/h. Assume a flat area and the passing is prohibited. Hint: deceleration rate is 3.5 (m/sec^2)



$$SSD=0.278\ V.t + \frac{v^2}{254\left[\left(\frac{a}{9.81}\right)\pm G\right]}$$

$$SSD=0.278 * 60*2.5 + \frac{60^2}{254\left[\left(\frac{3.5}{9.81}\right)\pm 0\right]}$$

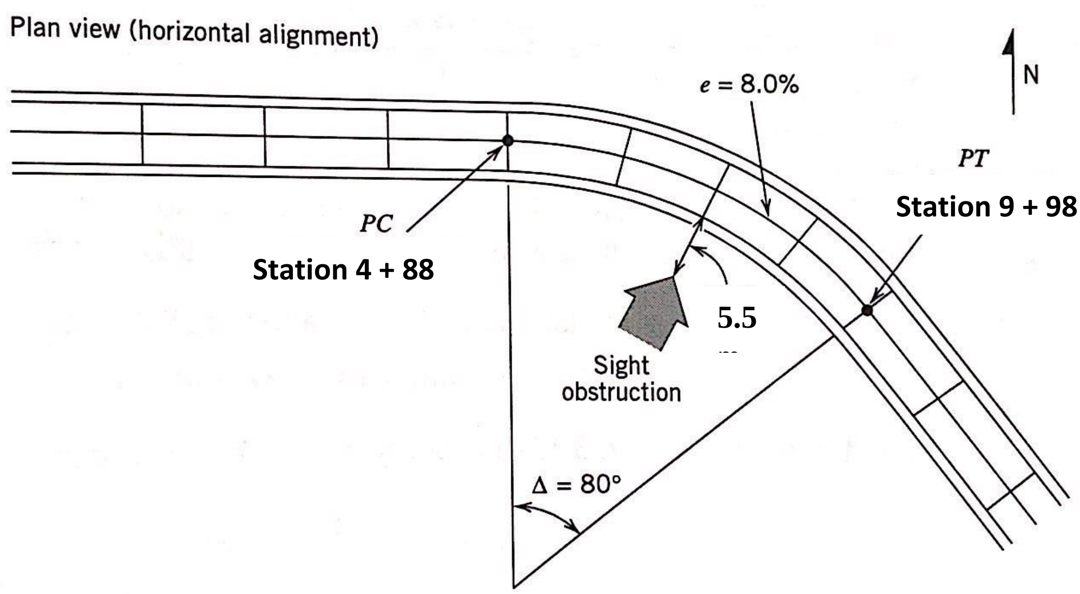
SSD = 81.4

$$m = 250 - (1\text{- } \cos \frac{28.65*81.4}{250}) = 3.34\text{m}$$

Q: *A horizontal curve is being designed through mountainous terrain for a four-ane with lanes that are 3 m wide. The central angle is known to be 40 degrees, the tangent distance is 155m, and the stationing of the tangent intersection (PI) is 8 + 23. Under specified conditions and vehicle speed, the roadway surface is determined to have a coefficient of side friction of 0.08, and the curve's superelevation is 0.09. what is the stationing of the PC and PT and what is the safe vehicle speed?*

Q: A new interstate highway is being built with a design speed of 110 km/h. For one of the horizontal curve, the radius is tentatively planned as 275m. What rate of superelevation is required for this curve?(hint: f =0.11 and e_{max}=8%)

Q: A two-lane highway (lane width of 3.6m) has a posted speed limit of 80 km/h and has horizontal curve as shown in the following figure. A recent daytime crash resulted in fatality and lawsuit alleging that the 80 Km/h posted speed limit is an unsafe speed for the curve in question and was a major cause of the crash. Evaluate and comment on the roadway design. Hint (the maximum side friction for a posted speed limit is 0.14 and the required standard stopping sight distance at 80 Km/h is 130m



1.6.5 Vertical alignment

Vertical alignment specifies the elevation of points along a roadway. The elevation of these roadway points are usually determined by the need to provide an acceptable level of driver safety, driver comfort and proper drainage. A primary concern in vertical alignment is establishing the transition of roadway elevations between two grades. This transition is achieved by means of a vertical curve. One of the most important factors that affect the design of this alignment is the topography of the area through which the proposed road is being passing as presented in Figures 1.30 and 1.31.

Vertical curves are usually parabolic in shape and can be broadly classified into crest vertical curves and sag vertical curves as illustrated in Figures 1.32 and 1.33.

Figure 1.30: Examples of vertical curves

Figure 1.31: Vertical curves in hilly areas

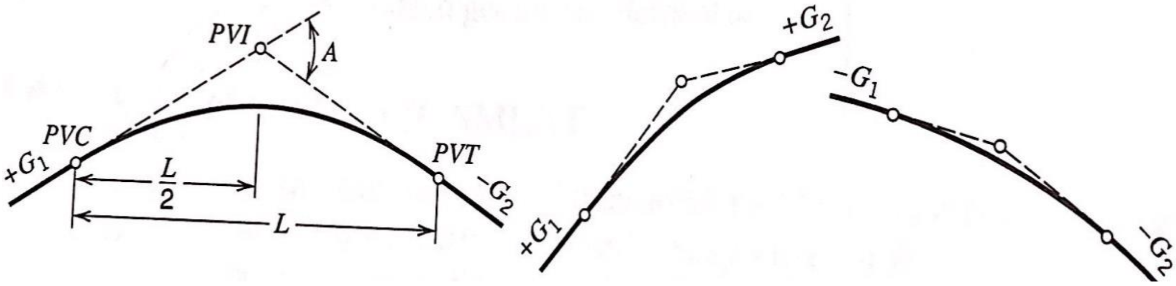


Figure 1.32: Crest vertical curves

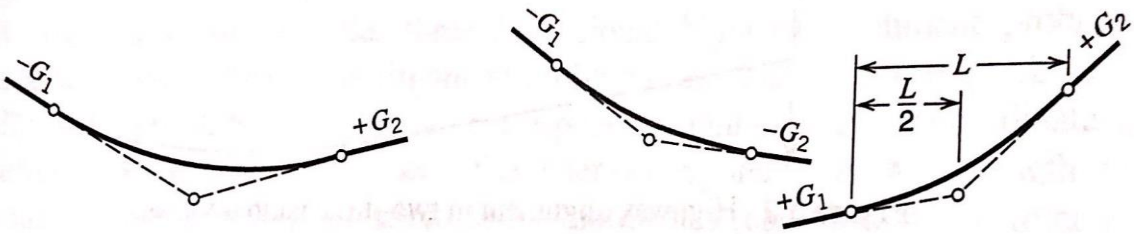


Figure 1.32: Sag vertical curves

1.6.5.1 Maximum grade

Passenger cars are normally less affected by the step grade as compared with the truck or heavy vehicle. Generally, the grade has a great effect on the heavy truck vehicles where a reduction of speed occurs on these grades. It should be noted that the selection of the grade value has a great influence on the volume of earthwork. To reduce this effect, it is customarily adopted to design the highways in such a way that ensure a reduction in the earthwork quantities and hence the cost of the project. Table 1.9 presents recommended maximum values of grades with respect to types of terrain and road.

1.6.5.2 Minimum grade

The minimum grade is generally governed by adopted drainage requirements for roadway being designed. A minimum grade of 0.3% is desirable for high type pavements.

1.6.5.3 Critical length of grade

This critical length can be defined as the maximum length of upgrade on which the design vehicle (almost heavy trucks) can run without a reasonable speed reduction. Figure 1.33 is used to assess the critical length of grade. It should be noted that a speed reduction curve of 15 Km/h is recommended to be used to find the critical length of grade.

Table 1.9: Recommended maximum value of grades

	Rural Collectors[a] Design Speed (mi/h)								
Type of Terrain	*20*	*25*	*30*	*35*	*40*	*45*	*50*	*55*	*60*
	Grades (%)								
Level	7	7	7	7	7	7	6	6	5
Rolling	10	10	9	9	8	8	7	7	6
Mountainous	12	11	10	10	10	10	9	9	8
	Urban Collectors[a] Design Speed (mi/h)								
Type of Terrain	*20*	*25*	*30*	*35*	*40*	*45*	*50*	*55*	*60*
	Grades (%)								
Level	9	9	9	9	9	8	7	7	6
Rolling	12	12	11	10	10	9	8	8	7
Mountainous	14	13	12	12	12	11	10	10	9
	Rural Arterials Design Speed (mi/h)								
Type of Terrain	*40*	*45*	*50*	*55*	*60*	*65*	*70*	*75*	*80*
	Grades (%)								
Level	5	5	4	4	3	3	3	3	3
Rolling	6	6	5	5	4	4	4	4	4
Mountainous	8	7	7	6	6	5	5	5	5
	Rural and Urban Freeways[b] Design Speed (mi/h)								
Type of Terrain	*50*	*55*	*60*	*65*	*70*	*75*	*80*		
	Grades (%)								
Level	4	4	3	3	3	3	3		
Rolling	5	5	4	4	4	4	4		
Mountainous	6	6	6	5	5	–	–		
	Urban Arterials Design Speed (mi/h)								
Types of Terrain	*30*	*35*	*40*	*45*	*50*	*55*	*60*		
	Grades (%)								
Level	8	7	7	6	6	5	5		
Rolling	9	8	8	7	7	6	6		
Mountainous	11	10	10	9	9	8	8		

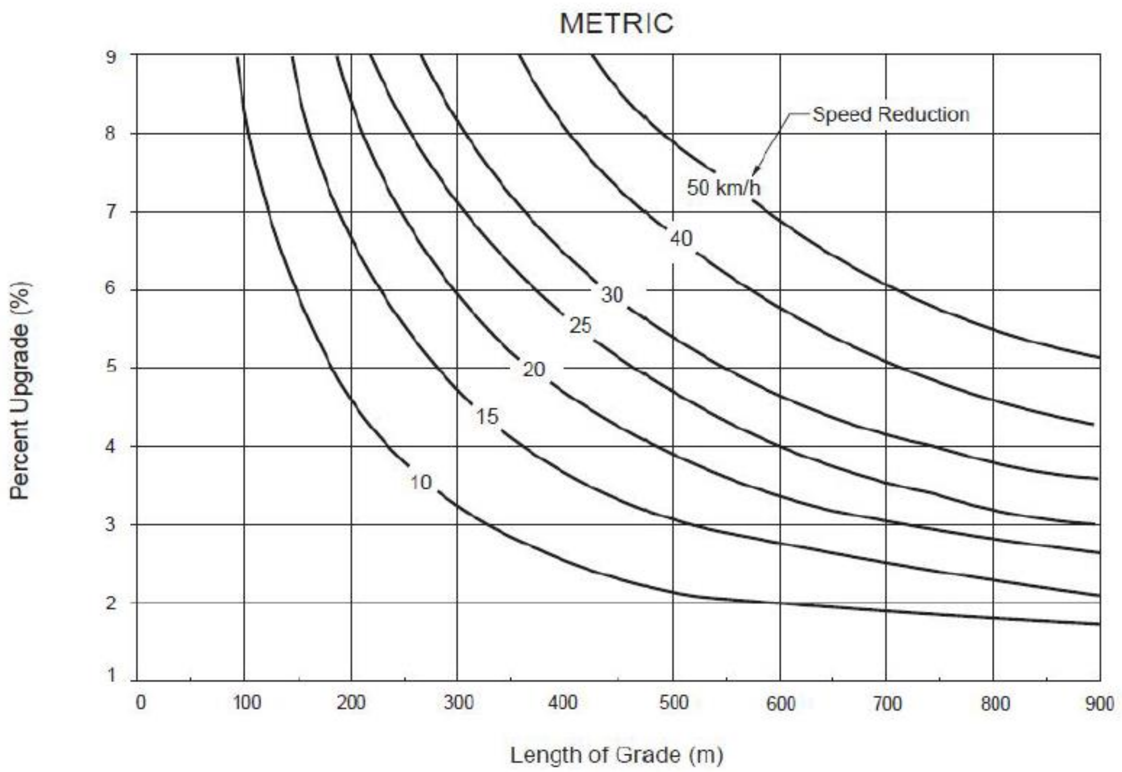


Figure 1.33: Critical length of grade

1.6.5.4 Elements of vertical curves

Elements of vertical curves can be illustrate in Figure 1.34

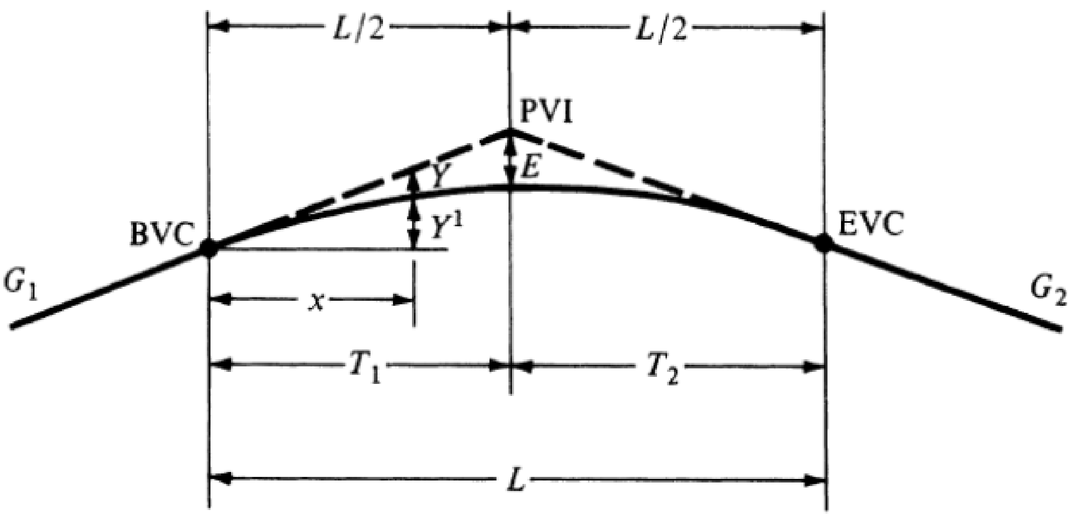


Figure 1.34: Layout and parameters of vertical curve

Where:

G_1, G_2: Grades of tangents %

L: Length of curve

E: External distance

BVC (PVC): beginning of vertical curve

EVC (PVT): End of vertical curve

PVI: point of vertical intersection

A: algebraic difference of grades, G_1-G_2

1.6.5.5 Properties of vertical curves

The determination of vertical curve elevations and elevation of critical points could be computed based on the properties of parabola as shown in equation

$$y = ax^2 + bx + c \quad \text{.......} 36$$

where

y = elevation of any point on curve.

x= distance from the point of vertical curvature.

a = rate of change of gradient.

b = initial grade

c= elevation of point of curvature

Rate of change of slope = the second derivative

First derivative = 2ax+b

Second derivative = 2a ……………………………………………………..37

But, the rate of change =$(G_2$-$G_1)$/100L ………………………………………..38

Equating Eq.37 and Eq.38 gives

$2a = (G_2-G_1)/100L$

So, $a = \frac{G2 - G1}{200L}$

And equation 36 can be rewritten as follows

Elevation of any point on curve= $\frac{G2-G1}{200\,L}\,x^2 + \frac{G1}{100}\,x$ +PVC elev.

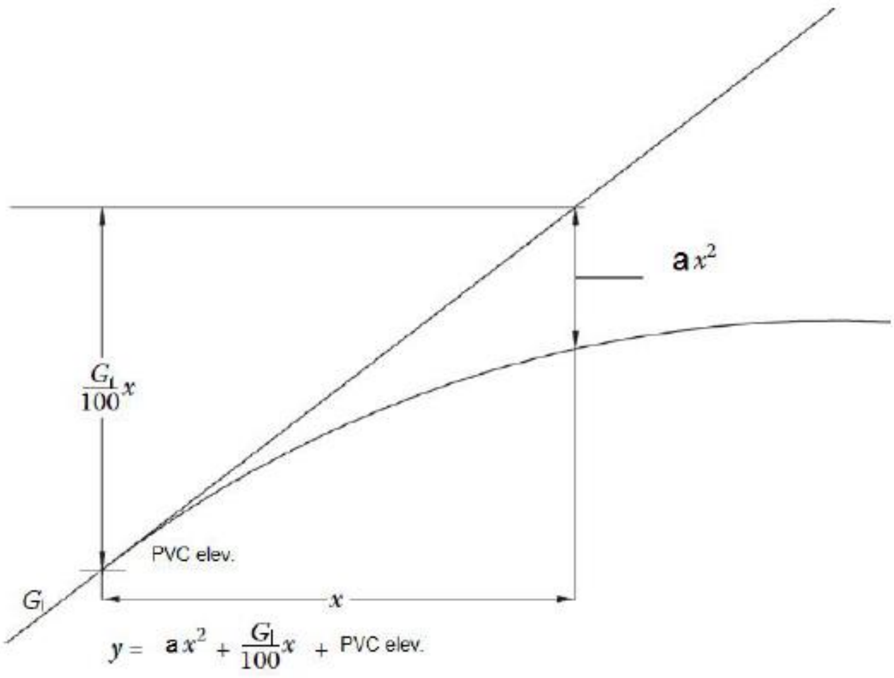


Figure 1.35: Layout and parameters of vertical curve

Offset

As shown in Figure 1.34, Y^1 can be calculated as follows:

$$Y^1 = \frac{G1}{100}X - Y \qquad \text{...39}$$

where $$Y = \frac{A}{200L}\,x^2 \qquad \text{...40}$$

A: algebraic difference of grades, G_1-G_2

$$Y^1 = \frac{G1}{100}X - \frac{G1-G2}{200L}\,x^2$$

$$\frac{dy1}{dx} = \frac{G1}{100} - \frac{G1-G2}{100L}\,x = 0$$

$$X_{\text{high/ low}} = \frac{G1}{G1-G2} L \quad \text{...41}$$

External distance *E* from the point of vertical intersection (PVI) to the curve is determined by substituting *L*/2 for *x* in Eq. $Y = \frac{A}{200L} x^2$

$$E = \frac{AL}{800} \quad \text{...42}$$

$$BVC_{\text{Station}} = PVI_{\text{station}} - \frac{L}{2} \quad \text{...43}$$

$$EVC_{\text{Station}} = BVC_{\text{station}} + L \quad \text{...44}$$

$$BVC_{\text{Elevation}} = PVI_{\text{Elevation}} - \frac{G_1 L}{200} \quad \text{...45}$$

$$EVC_{\text{Elevation}} = PVI_{\text{Elevation}} - \frac{G_2 L}{200} \quad \text{...46}$$

1.6.5.6 Design Procedure for Crest and Sag Vertical Curves

Step 1. Determine the minimum length of curve to satisfy sight distance requirements and other criteria for sag curves (sight distance requirements, comfort requirements. appearance requirements, and drainage requirements.

Step 2. Determine from the layout plans the station and elevation of the point where the grades intersect (PVI).

Step 3. Compute the elevations of the beginning of vertical curve, (BVC) and the end of vertical curve (EVC).

Step 4. Compute the offsets, *Y*, (Eq. 40) as the distance between the tangent and the curve. Usually equal distances of 20m (1 station) are used, beginning with the first whole station after the BVC.

Step 5. Compute elevations on the curve for each station.

Step 6. Compute the location and elevation of the highest (crest) or lowest (sag) point on the curve

1.6.5.7 Determine the minimum length of curve

When length of vertical curves needs to be computed, four scenarios/ criteria should be taken in account. Those includes:

1. Sight distance requirements.
2. Comfort requirements.
3. Appearance requirements.
4. Drainage requirements

The first criteria is only used to design the crest vertical curve; whereas all criteria are taken in account the process of design sag vertical curves.

1.6.5.7.1 Crest vertical Curves

As mentioned previously, crest vertical curves are commonly designed on the basis of sight distance requirements. Two scenarios exist and controls the design. These are when the length of curve is greater than the sight distance (L > S) and when the length of curve is less than the sight distance. Figure 1.36 shows the first case which is the more poplar or common design option.

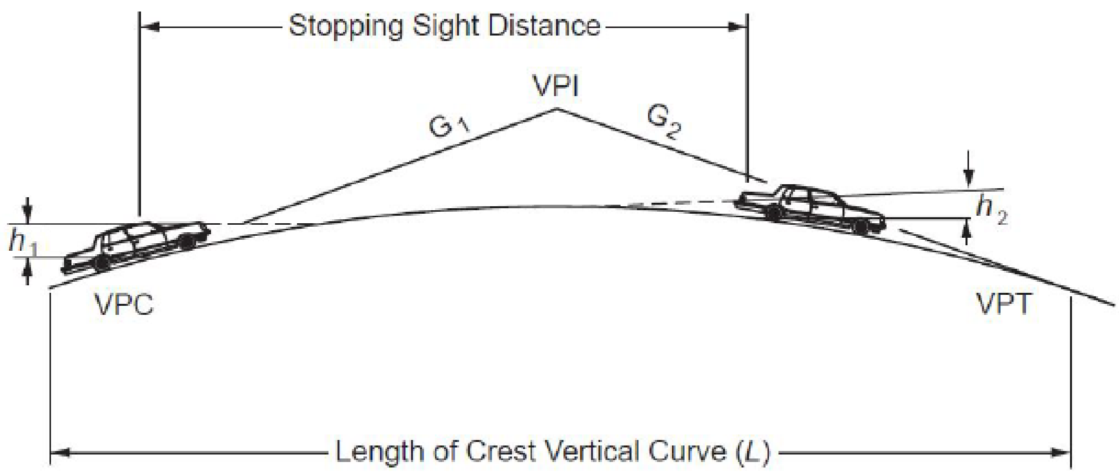


Figure 1.36: Crest vertical curves

The following equations are used to compute minimum length of vertical curve for both design option stated above:

When S is less than L

$$L_{min} = \frac{AS^2}{200\left(\sqrt{h1}+\sqrt{h2}\right)^2} \quad \ldots\ldots 47$$

When S is greater than L

$$L_{min} = 2S - \frac{200\left(\sqrt{h1}+\sqrt{h2}\right)^2}{A} \quad \ldots\ldots 48$$

Where:

L is length of vertical curve, *m*

A is algebraic difference in grades, *%*

S is sight distance, *m*

h1 is height of eye above roadway surface, *m*

h2 is height of object above roadway surface, *m*

Based on AASHTO's G.D policy, the values of h1 and h2 are 1.08 and 0.6 m, respectively. So by applying these values in equations above results, we get:

When S is less than L

$$L_{min} = \frac{AS^2}{658} \quad \ldots\ldots 49$$

When S is greater than L

$$L_{min} = 2S - \frac{658}{A} \quad \ldots\ldots 50$$

Design controls: stopping sight distance

Equation 49 (for S is less than L) can be rewritten as follows;

L= K. A ……………………………………………………………….51

Where;

$K = S^2 / 658$ ……………………………………………………………….52

And, K value represent the length of curve for each 1 degree change in the grade.

It should be noted in practice that when S > L, the calculated minimum length will be small and impractical for design consideration. Consequently, the designer should adopt minimum of crest vertical curve of L=0.6V (where L and V represent length of curve and design speed in Km/h, respectively) or use the first equation 49 to compute the design minimum length of curve. Figure 1.37 and Table 1.10 illustrate design controls for crest vertical curves based on stopping sight distance.

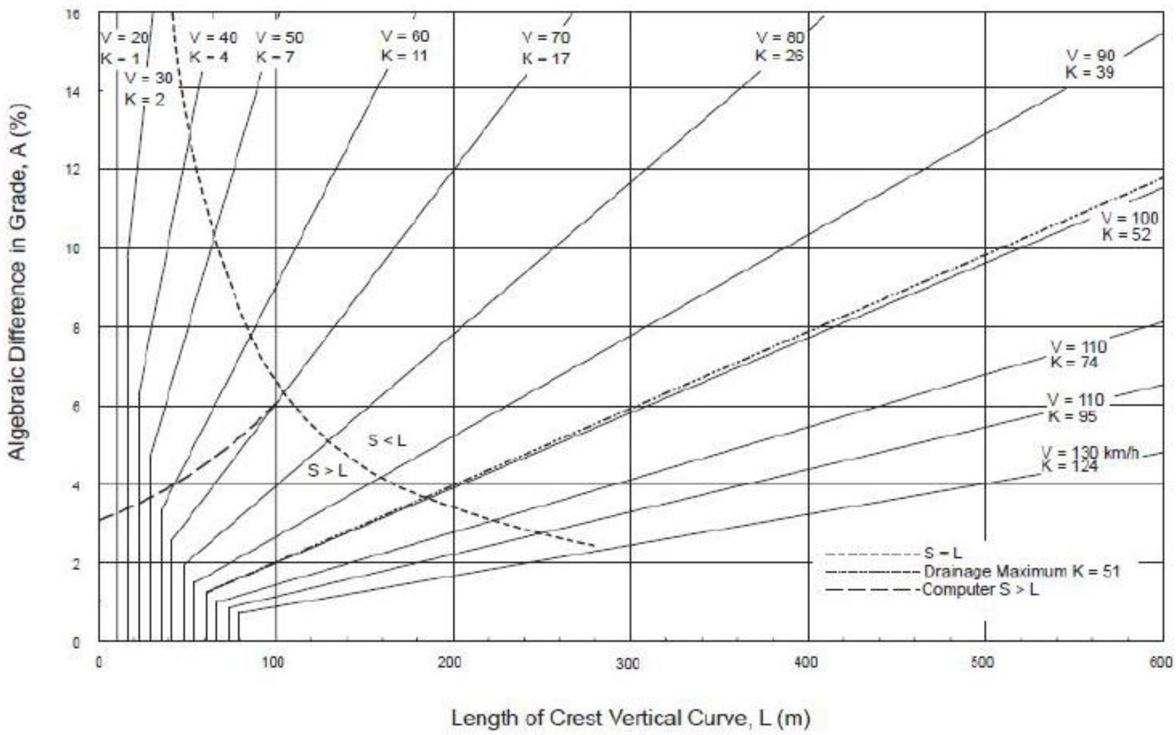


Figure 1.37: Design controls for crest curve

Table 1.10: Design controls for crest vertical curves based on stopping sight distance.

Metric				U.S. Customary			
Design Speed (km/h)	Stopping Sight Distance (m)	Rate of Vertical Curvature, K^a		Design Speed (mph)	Stopping Sight Distance (ft)	Rate of Vertical Curvature, K^a	
		Calculated	Design			Calculated	Design
20	20	0.6	1	15	80	3.0	3
30	35	1.9	2	20	115	6.1	7
40	50	3.8	4	25	155	11.1	12
50	65	6.4	7	30	200	18.5	19
60	85	11.0	11	35	250	29.0	29
70	105	16.8	17	40	305	43.1	44
80	130	25.7	26	45	360	60.1	61
90	160	38.9	39	50	425	83.7	84
100	185	52.0	52	55	495	113.5	114
110	220	73.6	74	60	570	150.6	151
120	250	95.0	95	65	645	192.8	193
130	285	123.4	124	70	730	246.9	247
				75	820	311.6	312
				80	910	383.7	384

[a] Rate of vertical curvature, *K*, is the length of curve per percent algebraic difference in intersecting grades

Design controls: passing sight distance

Based on AASHTO's G.D policy, both values of h1 and h2 (in case of passing sight distance application as shown in Figure 1.38) should be adopted as 1.08. By applying these values in equations 47 and 48, we get:

When S is less than L

$$L_{min} = \frac{AS^2}{864} \quad \ldots\ldots 53$$

When S is greater than L

$$L_{min} = 2S - \frac{864}{A} \quad \ldots\ldots 54$$

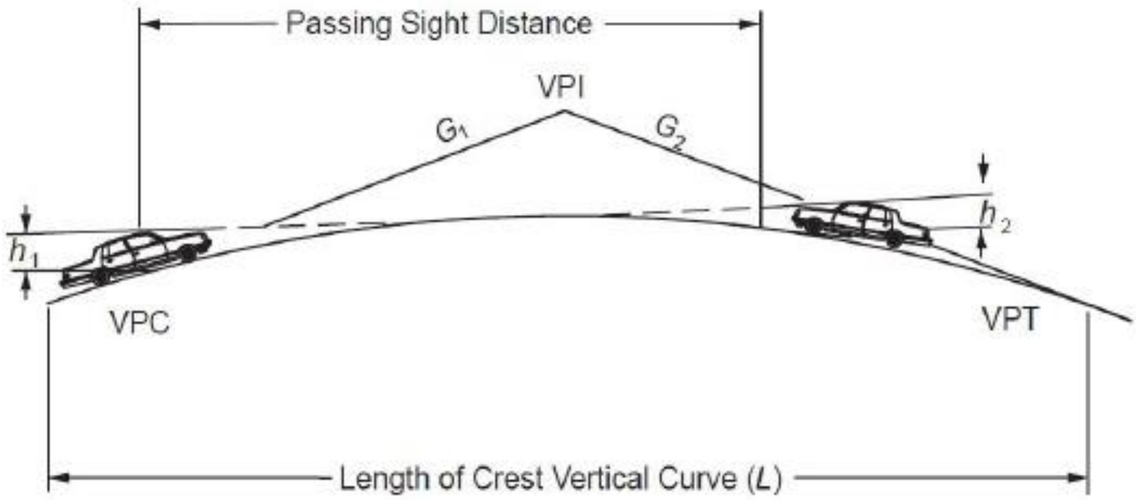


Figure 1.38: Passing sight distance on crest vertical

Table 1.11: Design controls for crest vertical curves based on passing sight distance

Metric			U.S. Customary		
Design Speed (km/h)	Passing Sight Distance (m)	Rate of Vertical Curvature, K^a Design	Design Speed (mph)	Passing Sight Distance (ft)	Rate of Vertical Curvature, K^a Design
30	120	17	20	400	57
40	140	23	25	450	72
50	160	30	30	500	89
60	180	38	35	550	108
70	210	51	40	600	129
80	245	69	45	700	175
90	280	91	50	800	229
100	320	119	55	900	289
110	355	146	60	1000	357
120	395	181	65	1100	432
130	440	224	70	1200	514
			75	1300	604
			80	1400	700

[a] Rate of vertical curvature, K, is the length of curve per percent algebraic difference in intersecting grades (A). $K = L/A$

1.6.5.7.2 Sag vertical Curves

Having mentioned that the minimum length of sag vertical curve is governed by four criteria, which include:

1. Sight distance requirements.
2. Comfort requirements.
3. Appearance requirements.
4. Drainage requirements

Sag curve minimum length based on sight distance requirements

Sight distance in this type of highways depends on the lighted part of the roadway ahead for the driver as shown in Figures 1.39. This is called as headlight sight distance as previously defined. On day time or on well-lit roadway at night, there is no problem with sight distance on this type of curves. Headlight sight distance is therefore mainly used by most highway department to estimate the length of the sag curve.

Figure 1.39: Sag vertical curve at day and night time

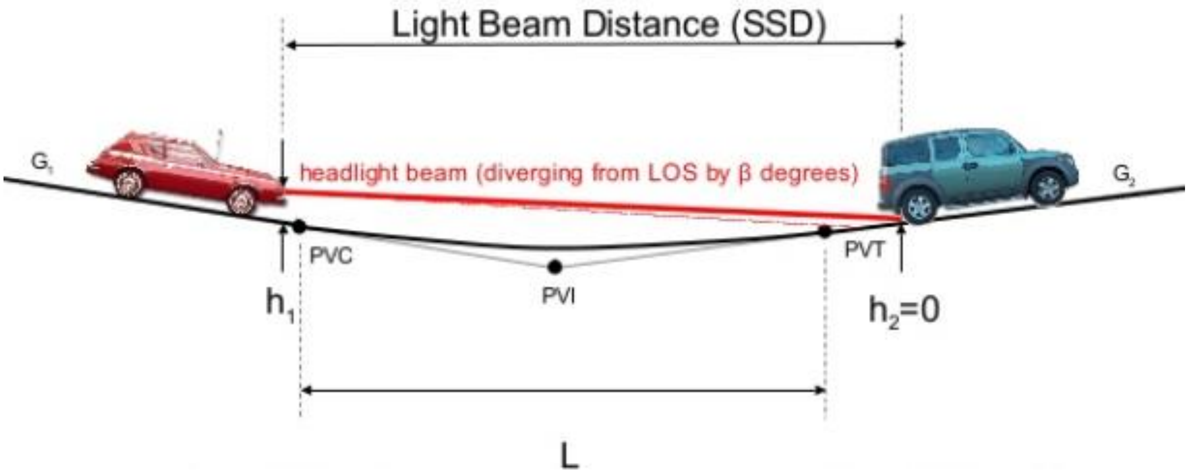


Figure 1.40: headlight (stopping) sight distance on crest vertical

According to sight distance requirement

When S is less than L

$$L_{min} = \frac{AS^2}{200(h + \tan\beta)} \quad \text{.......55}$$

When S is greater than L

$$L_{min} = 2S - \frac{200(h + \tan\beta)}{A} \quad \text{.......56}$$

Based on AASHTO's G.D policy, values of h and β are 0.6m and 1° respectively. And by applying these values, we get

When S is less than L

$$L_{min} = \frac{AS^2}{120 + 3.5\,S} \quad \text{.......57}$$

When S is greater than L

$$L_{min} = 2S - \frac{120 + 3.5\,S}{A} \quad \text{.......58}$$

Table 1.13 presents design controls for sag vertical curves based on stopping sight distance.

Sag curve minimum length based on driver comfort

Unlike on crest vertical curves, vehicle on sag curve is under a combination of gravitational and centrifugal forces. This combination may apply discomfort to the driver on this type of curves. To satisfy this criterion, the minimum length of curve should be estimated from the following formula.

$$L = \frac{AV^2}{395} \qquad \text{.........59}$$

Sag curve minimum length based on general appearance

Vertical curves are normally provided at all change in grade. However, for the slight change in grade (small A values), high K values are frequently provided to make sure that an appropriate appearance exist. Table 1.12 illustrates the maximum change in gradient that do not require a vertical curves and also the minimum length of curves for satisfactory appearance.

Table 1.12: Appearance requirement requirements

Design speed (km/h)	Maximum gradient change without vertical curve (%)	Minimum length of vertical curve for satisfactory appearance (m)
40	1.0	30
60	0.8	50
80	0.6	80
100	0.4	100
120	0.2	150

Sag curve minimum length based on drainage requirements

This criterion has to be considered in the case of curbed roads. In this scenario, the requirement is normally focuses on the maximum length whereas minimum lengths for other criteria are required. To satisfy this criterion, the maximum length should ensure that there is a minimum grade of 0.35 at the lowest 15 m of the curve. The maximum length to meet this requirement is normally equal the minimum length for other criterion for speed over 60 km/h.

***Q1**: An equal-tangent vertical curve is to be constructed between grades of -2% (initial) and +1.0% (final). The PVI is at station 3 +352.8 and at elevation 128.016m. Due to a street crossing the roadway, the elevation of the roadway at station 3+ 413.76 must be at 129.388m. Design the curve*

***Q2**: A vertical curve crosses a 1.219m diameter pipe at right angles. The pipe is located at station 3+ 378.708 and its centreline is at elevation 332.72m. The PVI of the vertical curve is at station 3+352.8 and elevation 334.792m. The vertical curve is equal tangent 182.88m long, and connects an initial grade of +1.2% and a final grade of -1.08%. Using offsets, determine the depth, below the surface of the curve, of the top of the pipe and determine the station of the highest point on the curve.*

***Q3:** A highway is being designed to AASHTO guidelines with 110 Km/h design speed, and at one section, an equal-tangent vertical curve must be designed to connect grades of +1.0% and -2.0%. Determine the minimum length of curve necessary to meet SSD requirements.*

***Q4**: A sag vertical curve joins a -3% grade and a +3% grade. If the PVI of the grades is at station 132+74.04 and has an elevation of 71.63m, determine station and elevation of the BVC and EVC for a design speed of 110Km/h. Also, compute the elevation on the curve at 20m intervals. (Hint: K value for design speed of 110Km/h is 55.*

Q5: *A crest vertical curve is to be designed to join a +3% grade with a -2% grade at a section of a two-lane highway. Determine the minimum length of the curve if the design speed of the highway is 100 Km/h, and a perception-reaction time of 2.5 sec. The deceleration rate for braking (a) is 3.5 m/sec^2.*

Q6*: An existing vertical curve on a highway joins a +4.4% grade with a -4.4% grade. If the length of the curve is 83.82m, what is the maximum safe speed on this curve? What speed should be posted if 5 mph increments are used? Assume a is 3.5 m/sec^2, perception-reaction time is 2.5 sec, and that Sight distance is less than length of vertical curve, L.*

Q7*: A sag vertical curve is to be designed to join a -5% grade to a +2% grade. If the design speed is 65 Km/h, determine the minimum length of the curve that will satisfy all criteria. Assume a is 3.5 m /sec^2 and perception-reaction time is 2.5 sec*

Chapter: Two

Pavement Design

2.1 Introduction

Pavement are among the costliest items associated with highway construction and maintenance, and are largely responsible for making highway system the most expensive public works project undertaken by any society. Because the pavement and associated shoulder structures are the most expensive items to construct and maintain, it is important for highway engineers to have a basic understanding of pavement design principles.

Highway pavement is a structure consisting of superimposed layers of processed materials above the natural soil sub-grade, whose primary function is to distribute the applied vehicle loads to the sub-grade. The pavement structure should be able to provide a surface of acceptable riding quality, adequate skid resistance, favourable light reflecting characteristics, and low noise pollution. The ultimate aim is to ensure that the transmitted stresses due to wheel load are sufficiently reduced, so that they will not exceed bearing capacity of the subgrade.

2.2 Requirements of Pavement

An ideal pavement should meet the following requirements:

- Sufficient thickness to distribute the wheel load stresses to a safe value on the sub-grade soil,
- Structurally strong to withstand all types of stresses imposed upon it,
- Adequate coefficient of friction to prevent skidding of vehicles,
- Smooth surface to provide comfort to road users even at high speed,
- Produce least noise from moving vehicles,
- Dust proof surface so that traffic safety is not impaired by reducing visibility.

- Impervious surface, so that sub-grade soil is well protected, and
- Long design life with low maintenance cost.

2.3 Factors affecting pavement design

2.3.1 Traffic and loading

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

- ***Contact pressure***: The tyre pressure is an important factor, as it determines the contact area and the contact pressure between the wheel and the pavement surface. Even though the shape of the contact area is elliptical, for sake of simplicity in analysis, a circular area is often considered.
- ***Wheel load***: The next important factor is the wheel load, which determines the depth of the pavement required to ensure that the subgrade soil is not failed. Wheel configuration affects the stress distribution and deflection within a pavement. Many commercial vehicles have dual rear wheels which ensure that the contact pressure is within the limits. The normal practice is to convert dual wheel into an equivalent single wheel load so that the analysis is made simpler.
- ***Axle configuration***: The load carrying capacity of the commercial vehicle is further enhanced by the introduction of multiple axles.
- ***Moving loads***: The damage to the pavement is much higher if the vehicle is moving at creep speed. Many studies show that when the speed is increased from 2 km/hr to 24 km/hr, the stresses and deflection reduced by 40%.
- ***Repetition of Loads***: The influence of traffic on pavement not only depends on the magnitude of the wheel load, but also on the frequency of

the load applications. Each load application causes some deformation and the total deformation is the summation of all these. Although the pavement deformation due to single axle load is very small, the cumulative effect of number of load repetition is significant. Therefore, modern design is based on total number of standard axle load (usually 80 KN single axle).

2.3.2 Structural models

The structural models are various analysis approaches to determine the pavement responses (stresses, strains, and deflections) at various locations in a pavement due to the application of wheel load. The most common structural models are layered elastic model and visco-elastic models.

2.3.3 Material characterization

The following material properties are important for both flexible and rigid pavements.

- When pavements are considered as linear elastic, the elastic moduli and Poisson ratio of subgrade and each component layer must be specified.
- If the elastic modulus of a material varies with the time of loading, then the resilient modulus, which is elastic modulus under repeated loads, must be selected in accordance with a load duration corresponding to the vehicle speed.
- When a material is considered non-linear elastic, the constitutive equation relating the resilient modulus to the state of the stress must be provided. However, many of these material properties are used in visco-elastic models which are very complex and in the development stage.

2.3.4 Environmental factors

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

- ***Temperature***. The effect of temperature on asphalt pavements is different from that of concrete pavements. Temperature affects the resilient modulus of asphalt layers, while it induces curling of concrete slab. In rigid pavements, due to difference in temperatures of top and bottom of slab, temperature stresses or frictional stresses are developed. While in flexible pavement, dynamic modulus of asphaltic mixture varies with temperature. Frost causes differential settlements and pavement roughness. Most detrimental effect of frost penetration occurs during the spring break up period when the ice melts and subgrade is in a saturated condition.
- ***Precipitation***. The precipitation from rain and snow affects the quantity of surface water infiltrating into the subgrade and the depth of ground water table. Poor drainage may bring lack of shear strength, pumping, loss of support, etc.

2.4 Pavement materials

2.4.1 Soil

Soil is an accumulation or deposit of earth material, derived naturally from the disintegration of rocks or decay of vegetation that can be excavated readily with power equipment in the field or disintegrated by gentle mechanical means in the laboratory. The supporting soil beneath pavement and its special under courses is called sub grade. Undisturbed soil beneath the pavement is called natural sub grade. Compacted sub grade is the soil compacted by controlled movement of heavy compactors. In general, the desirable properties of sub grade soil as a highway material are:

- Stability
- Incompressibility
- Permanency of strength

- Minimum changes in volume and stability under adverse conditions of weather and ground water
- Good drainage, and
- Ease of compaction

2.4.2 Aggregate

Aggregate is a collective term for the mineral materials such as sand, gravel, and crushed stone that are used with a binding medium (such as water, bitumen, Portland cement, lime, etc.) to form compound materials (such as bituminous concrete and Portland cement concrete). By volume, aggregate generally accounts for 92% to 96% of bituminous concrete and about 70% to 80% of Portland cement concrete. Aggregate is also used for base and sub-base courses for both flexible and rigid pavements. Aggregates can either be natural or manufactured. Natural aggregates are generally extracted from larger rock formations through an open excavation (quarry). Extracted rock is typically reduced to usable sizes by mechanical crushing. Manufactured aggregate is often a by-product of other manufacturing industries. Figure 2.1 illustrates some types of aggregate . Aggregate used in asphaltic mixtures are either:

1. **Crushed aggregate** (such as limestone, granite),
2. **Natural aggregate** (such as gravel and sand) or
3. **Secondary aggregate** (such as RAP, demolition aggregate, ...etc.)

Figure 2.1: Some types of aggregates

The main desirable properties of aggregate are:

- ***Strength.*** The aggregates used in top layers are subjected to (i) Stress action due to traffic wheel load, (ii) Wear and tear, (iii) crushing. For a high quality pavement, the aggregates should possess high resistance to crushing, and to withstand the stresses due to traffic wheel load.
- ***Hardness.*** The aggregates used in the surface course are subjected to constant rubbing or abrasion due to moving traffic. The aggregates should be hard enough to resist the abrasive action caused by the movements of traffic. The abrasive action is severe when steel tyre vehicles moves over the aggregates exposed at the top surface.
- ***Toughness***. Resistance of the aggregates to impact is termed as toughness. Aggregates used in the pavement should be able to resist the effect caused by the jumping of the steel tyre wheels from one particle to another at different levels which causes severe impact on the aggregates.
- ***Shape of aggregates***. Aggregates, which happen to fall in a particular size range, may have rounded, cubical, angular, flaky or elongated particles. It is evident that the flaky and elongated particles will have less strength and durability when compared with cubical, angular or rounded

particles of the same aggregate. Hence, too flaky and too much elongated aggregates should be avoided as far as possible.

- ***Adhesion with bitumen.*** The aggregates used in bituminous pavements should have less affinity with water when compared with bituminous materials; otherwise, the coated aggregated by bitumen will be stripped off in the presence of water.
- ***Durability***. The property of aggregates to withstand adverse action of weather is called soundness. The aggregates are subjected to the physical and chemical action of rain and bottom water, impurities there-in and that of atmosphere, hence it is desirable that the road aggregates used in the construction should be sound enough to withstand the weathering action.
- ***Freedom from deleterious particles.*** Specifications for aggregates used in bituminous mixes usually require the aggregates to be clean, tough, durable in nature, and free from excess amount of elongated pieces, dust, clay balls and other objectionable material. Similarly, aggregates used in Portland cement concrete mixes must be clean and free from deleterious substances such as clay lumps, silt and other organic impurities.

2.4.3 Bitumen

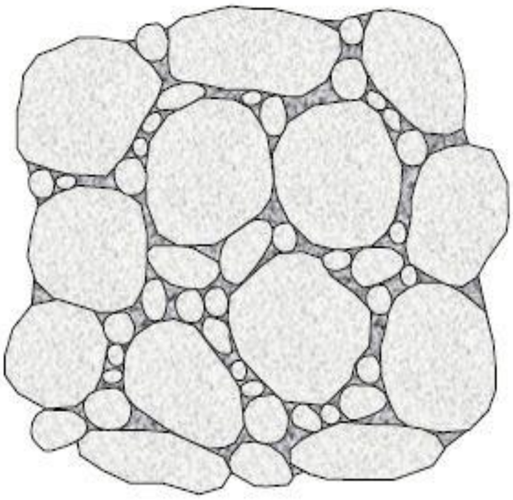
Bituminous materials are widely used all over the world in highway construction. These hydrocarbons are found in natural deposits or are obtained as a product of the distillation of crude petroleum. The bituminous materials used in highway construction are either asphalts or tars. All bituminous materials consist primarily of bitumen and have strong adhesive properties with colours ranging from dark brown to black. They vary in consistency from liquid to solid; thus, they are divided into liquids, semisolids, and solids. The solid form is usually hard and brittle at normal temperatures but will flow when subjected to long, continuous loading. The liquid form is obtained from the semisolid or solid forms by heating, dissolving in solvents, or breaking the material into minute particles and dispersing them in water with an emulsifier to form an asphalt emulsion. Figure 2.2 illustrates bitumen

Figure 2.2: Bitumen

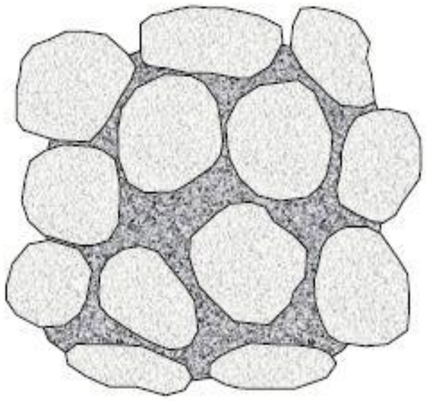
2.5 Asphalt Mixtures

2.5.1 Mixtures gradation types

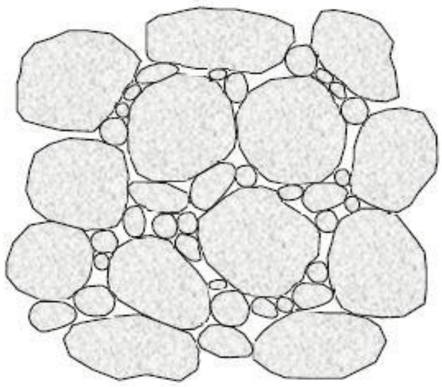
- ***Dense Graded Aggregate or Well Graded Aggregate***

Illustration	*Characteristics*
	• Wide range of sizes • Grain to grain contact • Low void content • Low permeability • High stability • Difficult to compact

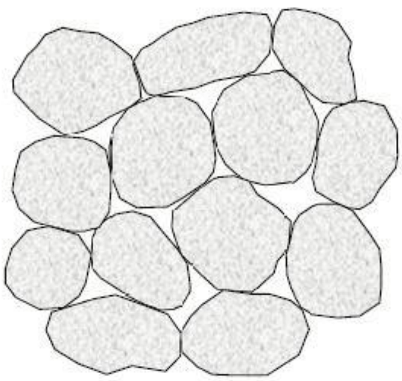
- ***Gap Graded Aggregate***

Illustrstion	***Characteristics***
	• Missing middle sizes • No Grain to grain contact • Moderate void content • Moderate permeability • Low stability • Easy to compact

- ***Open Graded Aggregate***

Illustration	***Characteristics***
	• Few fine particles • Grain to grain contact • High void content • High permeability • High stability • Difficult to compact

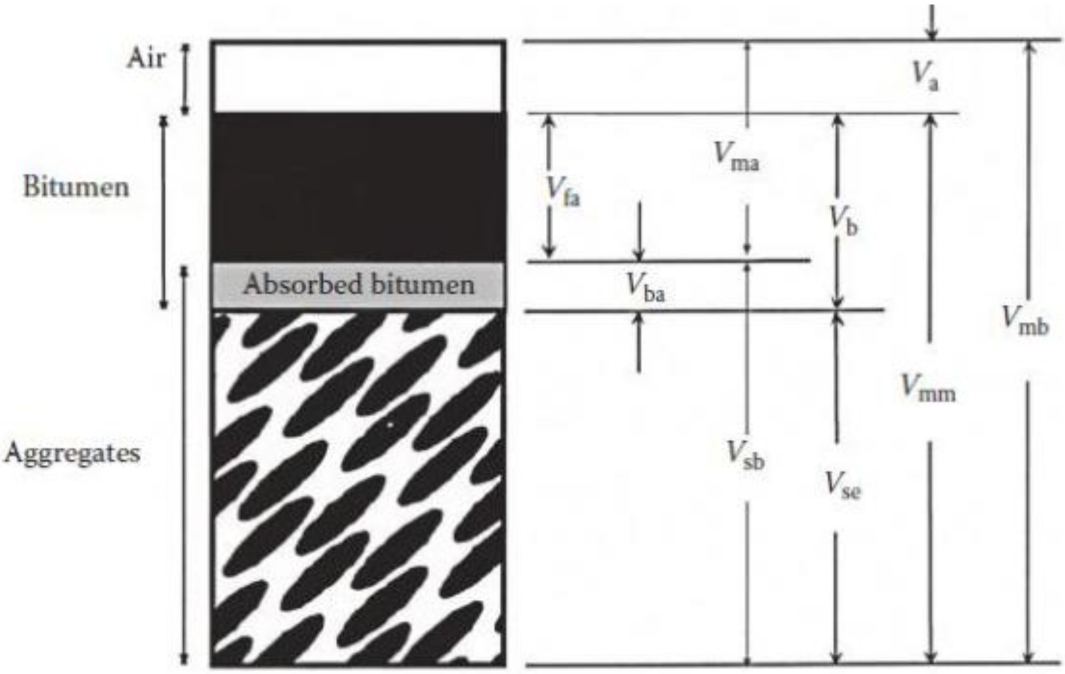
- ***Uniformly Graded Aggregate***

Illustration	***Characteristics***
	• Narrow range of sizes • Grain to grain contact • High void content • High permeability • Low stability • Difficult to compact

2.5.2 Compacted bituminous mixture

The volume of the compacted specimen of any bituminous mixture consists of the volume occupied by aggregates, the volume occupied by bitumen and the volume of air voids. The volume, which is occupied by bitumen and air voids, is known as volume in mineral aggregates (VMA). When bituminous binder is added, part of the volume of air voids is filled with bitumen (asphalt). The volume is known as voids filled with asphalt (VFA). The above volumetric characteristic properties are presented in Figure 2.3. Aggregates (attributed to the surface pores) normally possess and absorb a certain quantity of bitumen. As a consequence, the remaining bitumen quantity is in fact the one that coats the aggregates, fills the voids and provides cohesion in the mixture. This quantity of bitumen is designated as 'effective' bitumen quantity, and it is always less than the initial quantity of bitumen added, unless the aggregate's absorption is zero (ideal case).

The surface pores, in the absence of bitumen, absorb water (surface voids permeable to water). Because of the lower viscosity of water in comparison to bitumen's viscosity, water absorption is always higher than bitumen absorption. The schematic representation of an aggregate-coated particle given in Figure 2.4 the above, as well as other concepts.



V_{mb} = volume of compacted mix
V_a = volume of air voids
V_b = volume of bitumen
V_{ma} = volume of voids in mineral aggregates (VMA)
V_{sb} = volume of mineral aggregate (by bulk specific gravity)
V_{se} = volume of mineral aggregate (by effective specific gravity)
V_{fa} = voids filled with asphalt (VFA)
V_{mm} = voidless volume of paving mix
V_{ba} = volume of absorbed bitumen

Figure 2.3: Illustration of volumes in compacted asphaltic mixture

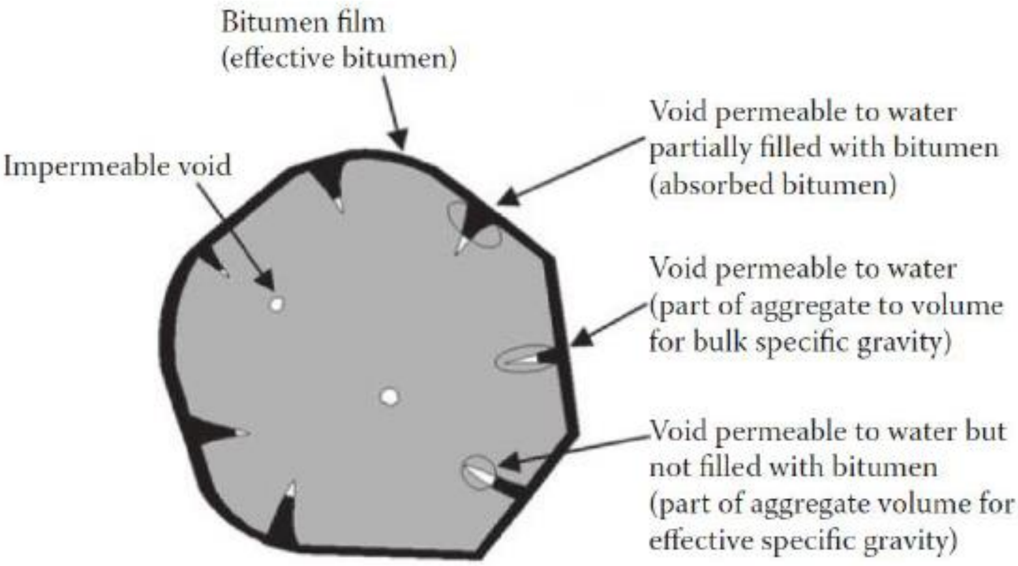


Figure 2.4: Coated aggregate particle

Provided that the bulk specific gravity (G_{sb}) and the effective specific gravity (G_{se}) of the total aggregate, as well as the specific gravity of the bitumen (G_b), are known, the volumetric properties of bituminous mixture (asphalt mixture) may be determined. The bulk density of the sample usually is determined by weighing the sample in air and in water. It may be necessary to coat samples made from open-graded mixtures with paraffin before determining the density. The bulk specific gravity G_{mb} of the sample—that is, the compacted mixture—is given as

$$G_{mb} = \frac{W_a}{W_a - W_w} \qquad \text{...(2.1)}$$

where

W_a = weight of sample in air (g)

W_w = weight of sample in water (g)

2.5.3 Determination of the design asphalt content of the mix

Analysis of Results from Marshall Test

The first step in the analysis of the results is the determination of the average bulk specific gravity for all test specimens having the same asphalt content. **The average unit weight of each mixture is then obtained by multiplying its average specific gravity by the density of water**. A smooth curve that represents the best fit of plots of unit weight versus percentage of asphalt is determined, as shown in Figure 2.5(a). This curve is used to obtain the bulk specific gravity values that are used in further computations as in Example 1.

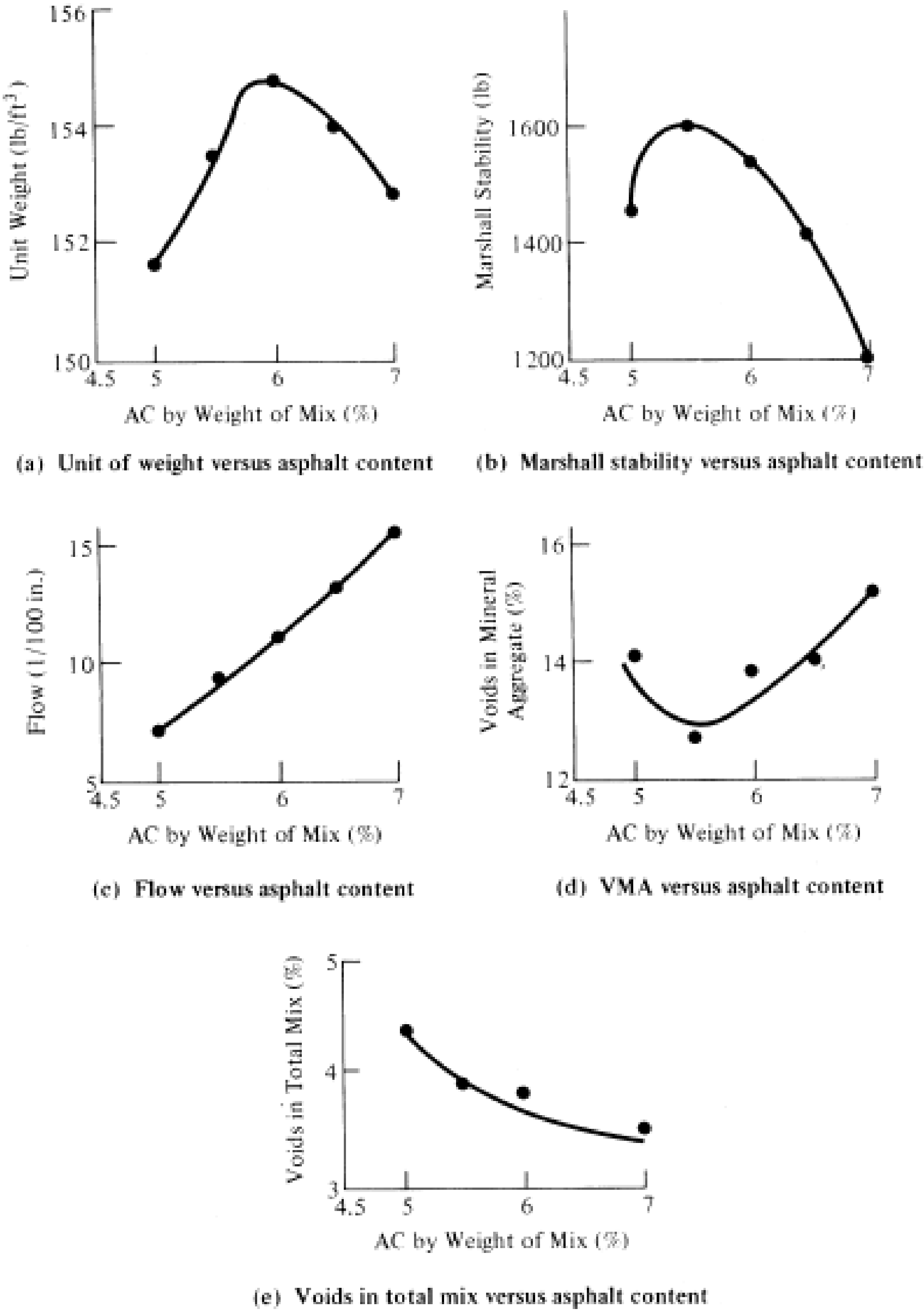


Figure 2.5: Determination of optimum binder content (OBC) using volumetric properties

In order to compute the percent air voids, the percent voids in the mineral aggregate, and the absorbed asphalt in pounds of the dry aggregate, it is first necessary to compute the **bulk specific gravity of the aggregate mixture, the apparent specific gravity of the aggregate mixture, the effective specific gravity of the aggregate mixture, and the maximum specific gravity of the paving mixtures for different asphalt contents**. These different measures of the specific gravity of the aggregates take into consideration the variation with which mineral aggregates can absorb water and asphalt.

Bulk Specific Gravity of Aggregate. The bulk specific gravity is defined as the weight in air of a unit volume (including all normal voids) of a permeable material at a selected temperature, divided by the weight in air of the same volume of gas-free distilled water at the same selected temperature. Since the aggregate mixture consists of different fractions of coarse aggregate, fine aggregate, and mineral fillers with different specific gravities, the bulk specific gravity of the total aggregate in the paving mixture is given as:

$$G_{sb} = \frac{P_{ca} + P_{fa} + P_{mf}}{\dfrac{P_{ca}}{G_{bca}} + \dfrac{P_{fa}}{G_{bfa}} + \dfrac{P_{mf}}{G_{bmf}}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2.2)$$

where:

G_{sb} = bulk specific gravity of aggregates in the paving mixture

P_{ca}, P_{fa}, P_{mf} = percent by weight of coarse aggregate, fine aggregate, and mineral filler, respectively, in the paving mixture. (Note that P_{ca}, P_{fa}, and P_{mf} could be found either as a percentage of the paving mixture or as a percentage of only the total aggregates. The same results will be obtained for G_{sb})

$G_{bca}, G_{bfa}, G_{bmf}$ = bulk specific gravities of coarse aggregate, fine aggregate, and mineral filler, respectively

It is not easy to accurately determine the bulk specific gravity of the mineral filler. The apparent specific gravity may therefore be used with very little error.

$$G_{asb} = \frac{P_{ca} + P_{fa} + P_{mf}}{\frac{P_{ca}}{G_{aca}} + \frac{P_{fa}}{G_{afa}} + \frac{P_{mf}}{G_{amf}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2.3)$$

where:

G_{asb} = *apparent* specific gravity of the aggregate mixture
P_{ca}, P_{fa}, P_{mf} = percent by weight of coarse aggregate, fine aggregate, and mineral filler, respectively, in the mixture
$G_{aca}, G_{afa}, G_{amf}$ = apparent specific gravities of coarse aggregate, fine aggregate, and mineral filler, respectively

Effective Specific Gravity of Aggregate. The effective specific gravity of the aggregate exception of those that are filled with asphalt. It is given as:

$$G_{se} = \frac{100 - P_b}{(100/G_{mm}) - (P_b/G_b)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2.4)$$

Where:

G_{se} = effective specific gravity of the aggregates
G_{mm} = maximum specific gravity of paving mixture (no air voids)
P_b = asphalt percent by total weight of paving mixture (thus $100 - P_b$ is the percent by weight of the base mixture that is not asphalt)
G_b = specific gravity of the asphalt

Maximum Specific Gravity of the Paving Mixture. The maximum specific gravity of the paving mixture G_{mm} assumes that there are no air voids in the asphalt concrete. Although the G_{mm} can be determined in the laboratory by conducting the standard test (ASTM Designation D2041), the best accuracy is attained at mixtures near the optimum asphalt content. Since it is necessary to determine the G_{mm} for all samples, some of which contain much lower or much higher quantities than the optimum asphalt content, the following procedure can be used to determine the G_{mm} for each sample.

The ASTM Designation D2041 test is conducted on all specimens containing a selected asphalt cement content and the mean of these is determined. This value is then used to determine the effective specific gravity of the aggregates using equation above. The effective specific gravity of the aggregates can be considered constant, since varying the asphalt content in the paving mixture does not significantly vary the asphalt absorption. The effective specific gravity obtained then is used to determine the maximum specific gravity of the paving mixtures with different asphalt cement contents using below

$$G_{mm} = \frac{100}{(P_s/G_{se}) + (P_b/G_b)} \qquad \text{……………………………..,,,,,,,,(2.5)}$$

where:

G_{mm} = maximum specific gravity of paving mixture (no air voids)
P_s = percent by weight of aggregates in paving mixture
P_b = percent by weight of asphalt in paving mixture
G_{se} = effective specific gravity of the aggregates (assumed to be constant for different asphalt cement contents)
G_b = specific gravity of asphalt

Once these different specific gravities have been determined, the asphalt absorption, the effective asphalt content, the percent voids in mineral aggregates (VMA), and the percent air voids in the compacted mixture all can be determined.

Asphalt absorption is the percent by weight of the asphalt that is absorbed by the aggregates based on the total weight of the aggregates. This is given as

$$P_{ba} = 100 \frac{G_{se} - G_{sb}}{G_{sb} G_{se}} G_b \qquad \text{...(2.6)}$$

Where:

P_{ba} = amount of asphalt absorbed as a percentage of the total weight of aggregates
G_{se} = effective specific gravity of the aggregates
G_{sb} = bulk specific gravity of the aggregates
G_b = specific gravity of asphalt

Effective Asphalt Content. The effective asphalt content is the difference between the total amount of asphalt in the mixture and that absorbed into the aggregate particles. The effective asphalt content is therefore that which coats the outside of the aggregate particles and influences the pavement performance. It is given as

$$P_{be} = P_b - \frac{P_{ba}}{100} P_s \qquad \text{...(2.7)}$$

Where:

P_{be} = effective asphalt content in paving mixture (percent by weight)
P_b = percent by weight of asphalt in paving mixture
P_s = aggregate percent by weight of paving mixture
P_{ba} = amount of asphalt absorbed as a percentage of the total weight of aggregates

Percent Voids in Compacted Mineral Aggregates. The percent voids in compacted mineral aggregates (VMA) is the percentage of void spaces between the granular particles in the compacted paving mixture, including the air voids and the volume occupied by the effective asphalt content. It usually is calculated as a percentage of the bulk volume of the compacted mixture based on the bulk specific gravity of the aggregates. It is given as:

$$VMA = 100 - \frac{G_{mb} P_s}{G_{sb}} \qquad \text{……………………………………..(2.8)}$$

Where;

VMA = percent voids in compacted mineral aggregates (percent of bulk volume)
G_{mb} = bulk specific gravity of compacted mixture
G_{sb} = bulk specific gravity of aggregate
P_s = aggregate percent by weight of total paving mixture

***Percent Air Voids in Compacted Mixture*.** This is the ratio (expressed as a percentage) between the volume of the small air voids between the coated particles and the total volume of the mixture. It can be obtained from below equation

$$P_a = 100 \frac{G_{mm} - G_{mb}}{G_{mm}} \qquad \text{………………………………………...(2.9)}$$

where:

P_a = percent air voids in compacted paving mixture
G_{mm} = maximum specific gravity of the compacted paving mixture
G_{mb} = bulk specific gravity of the compacted paving mixture

Four additional separate smooth curves are drawn: percent voids in total mix versus percent of asphalt, percent voids in mineral aggregate versus percent of asphalt, Marshall stability versus percent of asphalt, and flow versus percent of asphalt. These graphs are used to select the asphalt contents for maximum stability, maximum unit weight, and percent voids in the total mix within the limits specified (usually the median of the limits). The average of the asphalt contents is the optimum asphalt content. The stability and flow for this optimum content then can be obtained from the appropriate graphs to determine whether the required criteria are met. AASHTO suggested criteria for these test limits are given in Table 2.1. It should be noted that all criteria should be satisfied and not just the criterion for stability.

An example of design an optimum binder content is given separately.

Table 2.1: Marshall mix criteria

(a) Maximum and Minimum Values			
Marshall Method Mix Criteria	*Light Traffic ESAL < 10^4 (see Chapter 19)*	*Medium Traffic 10^4 < ESAL < 10^6 (see Chapter 19)*	*Heavy Traffic ESAL > 10^6 (see Chapter 19)*
Compaction (No. of blows each end of Specimen)	35	50	75
Stability *N* (lb)	3336 (750)	5338 (1200)	8006 (1800)
Flow, 0.25 mm (0.1 in.)	8 to 18	8 to 16	8 to 14
Air Voids (%)	3 to 5	3 to 5	3 to 5

(b) Mineral Percent Voids in Mineral Aggregates

Standard Sieve Designation	*Percent*
No. 16	23.5
No. 4	21
No. 8	18
⅜ in.	16
½ in.	15
¾ in.	14
1 in.	13
1½ in.	12
2 in.	11.5
2½ in.	11

SOURCE: Federal Highway Administration, U.S. Department of Transportation.

2.6 Pavement Types

In general, there are two types of pavement structures: flexible pavements and rigid pavements. There are however, many variations of these pavements types, including some with soil cement and stabilized bases that have cemented aggregate. Composite pavements (which are made of both rigid and flexible layers), continuously reinforced pavements, and post-tensioned pavements (pre-cast) are other types, which are usually, require specialized design and are not covered in this stage.

As with any structure, the underlying soil must ultimately carry the load that is placed on it. Having mentioned that a pavement function is to distribute the traffic load stresses to the soil (sub-grade) at a magnitude that will not shear or distort the soil. Typical soil-bearing capacities can be less than 345 kPa and in some cases as low as 14 to 21 kPa. When soil is saturated with water, the bearing capacity can be very low, and in these cases, it is very important for pavement to distribute tires loads to the soil in such a way as to prevent failure of the pavement structure. Figure 2.6 shows a difference in stress distribution through flexible and rigid pavements.

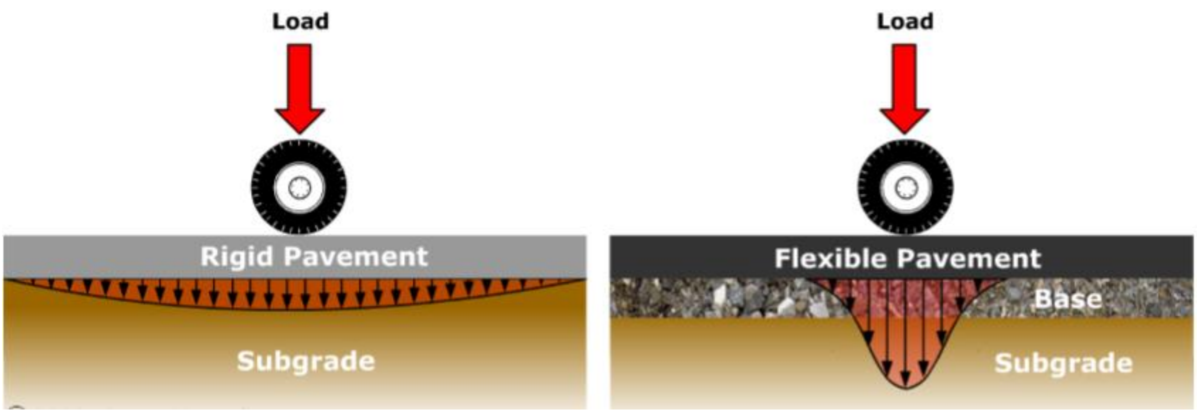


Figure 2.6: Stresses distribution under rigid and flexible pavements

In general, Table 2.2 illustrate the key points difference between flexible and rigid pavements while Figures 2.6 and 2.7 show the structure layers of flexible and rigid pavements respectively.

Table 2.2: Key points difference between flexible and rigid pavements

Flexible Pavements	***Rigid Pavements***
It consists of a series of layers with the highest quality materials at or near the surface of pavements	*It consists of one layer Portland concrete slab or relatively high flexural strength*
It reflects the deformation of sub-grade and subsequently layers on the surface	*It is able to bridge over localized failures and area of inadequate support*
Its stability depends upon the aggregate interlock, particles friction and cohesion	*Its structural strength is provided by the pavement slab itself and by its beam action*
Pavement design is greatly influenced by the sub-grade strength	*Flexural strength of concrete is a major for design*
It functions by a way of load distribution through the component layers	*It distributes load over a wide area of sub-grade because of its rigidity and high modulus of elasticity*
Temperature variations due to change in atmospheric conditions do not produce stresses in flexible pavements	*Temperature changes induce heavy stresses in rigid pavements*
It has self-healing properties due to heavier wheel load and therefore it is recoverable in some extent	*Any excessive deformations due to heavier wheel loads are not recoverable. For example, settlements are permanent*

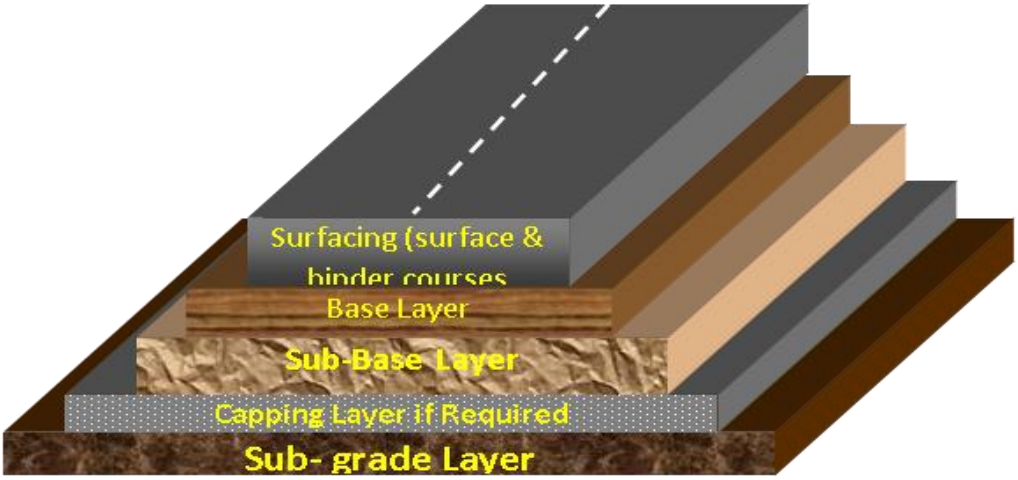


Figure 2.6: Typical structure of flexible pavements

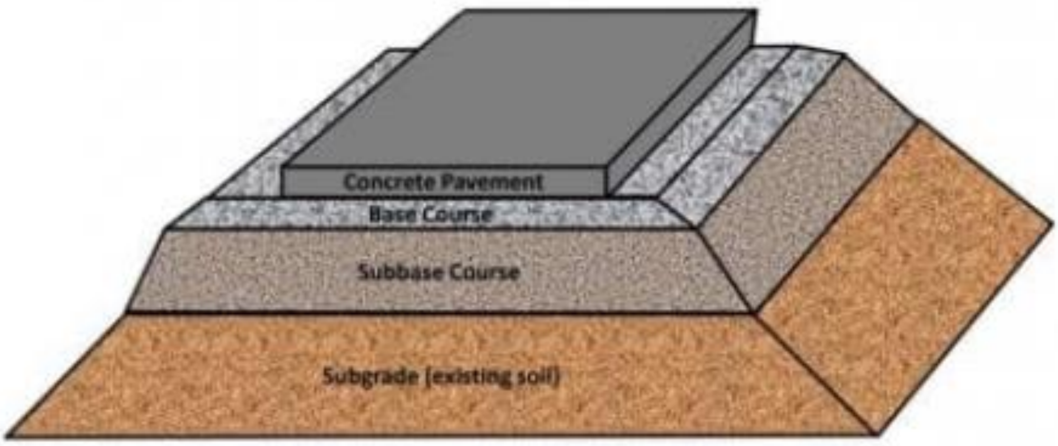


Figure 2.7: Typical structure of rigid pavements

2.6.1 Flexible Pavement

2.6.1.1 Types of Flexible Pavement

The following types of construction have been used in flexible pavement:

- Conventional layered flexible pavement.
- Full - depth asphalt pavement.
- Contained rock asphalt mat (CRAM).

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

Full - depth asphalt pavements are constructed by placing bituminous layers directly on the soil sub-grade. This is more suitable when there is high traffic and local materials are not available.

Contained rock asphalt mats are constructed by placing dense/open graded aggregate layers in between two asphalt layers. Modified dense graded asphalt concrete is placed above the sub-grade will significantly reduce the vertical compressive strain on soil sub-grade and protect from surface water.

2.6.1.2 Typical layers of Flexible Pavement

Typical layers of a conventional flexible pavement includes seal coat, surface course, tack coat, binder course, prime coat, base course, sub-base course, compacted sub-grade, and natural sub-grade. Figure 2.8 shows typical flexible pavement structure.

Seal Coat:

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

Tack Coat:

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

Prime Coat:

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

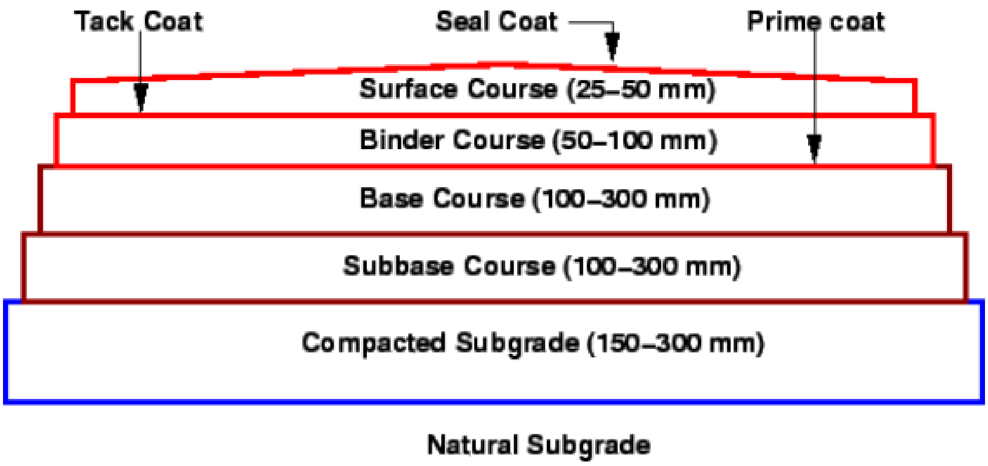


Figure 2.8: Typical flexible pavement structure.

Sub-grade:

The subgrade is usually the natural material located along the horizontal alignment of the pavement and serves as the foundation of the pavement structure. It also may consist of a layer of selected borrow materials, well compacted to prescribed specifications. It may be necessary to treat the subgrade material to achieve certain strength properties required for the type of pavement being constructed. Soil stabilization is the treatment of natural soil to improve its engineering properties. One solution to enhance the properties of sub-grade is to stabilize this layer. Soil stabilization methods can be divided into two categories, namely, mechanical and chemical. This can be achieved using one of these methods below;

1. ***Cement-stabilized soil*** is a mixture of water, soil, and measured amounts of Portlandcement—thoroughly mixed and compacted to a high density and then allowed to cure for a specific period, during which it is protected from loss of moisture.
2. ***Soil cement*** is a hardened material obtained by mechanically compacting a mixture of finely crushed soil, water, and a quantity of Portland cement that will make the mixture meet certain durability requirements.

3. ***Cement-modified soil*** is a semi hardened or unhardened mixture of water, Portland cement, and finely crushed soil. This mixture has less cement than the soil–cement mixture.
4. ***Plastic soil cement*** is a hardened material obtained by mixing finely crushed soil, Portland cement, and a quantity of water, such that at the time of mixing and placing, a consistency similar to that of mortar is obtained.
5. ***Soil-lime*** is a mixture of lime, water, and fine-grained soil. If the soil contains silica and alumina, pozzolanic reaction occurs, resulting in the formation of a cementing-type material. Clay minerals, quartz, and feldspars are all possible sources of silica and alumina in typical fine-grained soils.

Sub-Base Course:

The sub-base course is the layer of material beneath the base course and the primary functions are to provide structural support, improve drainage, and reduce the intrusion of fines from the sub-grade in the pavement structure. If the base course is open graded, then the sub-base course with more fines can serve as a filler between sub-grade and the base course. A sub-base course is not always needed or used. For example, a pavement constructed over a high quality, stiff sub-grade may not need the additional features offered by a sub-base course. In such situations, sub-base course may not be provided.

Base Course:

The base course is the layer of material immediately beneath the surface of binder course and it provides additional load distribution and contributes to the sub-surface drainage. It may be composed of crushed stone, crushed slag, and other untreated or stabilized materials.

Surface Course:

The surface course is the upper course of the road pavement and is constructed immediately above the base course. The surface course in flexible pavements usually consists of a mixture of mineral aggregates and asphalt. It should be capable of:

- Withstanding high tire pressures,
- Resisting abrasive forces due to traffic,
- Providing a skid resistant driving surface, and
- Preventing the penetration of surface water into the underlying layers.

The thickness of the wearing surface can vary from 75mm to more than 150 mm, depending on the expected traffic on the pavement. It should be noted that the quality of the surface course of a flexible pavement depends on the mix design of the asphalt concrete used.

2.6.1.3 Principle for flexible pavement

The primary function of the pavement structure is to reduce and distribute the surface stresses (contact tire pressure) to an acceptable level at the sub-grade (to a level that prevents permanent deformation). A flexible pavement reduces the stresses by distributing the traffic wheel loads over greater and greater areas, through the individual layers, until stress at the sub-grade is at an acceptable low level. The traffic loads are transmitted to the sub-grade by aggregate-to-aggregate particle contact. Confining pressures (lateral forces due to material weight) in the sub-base and base layers increase the bearing strength of these materials. A cone distribution loads reduces and spreads the stress to sub-grade as shown in Figure 2.9.

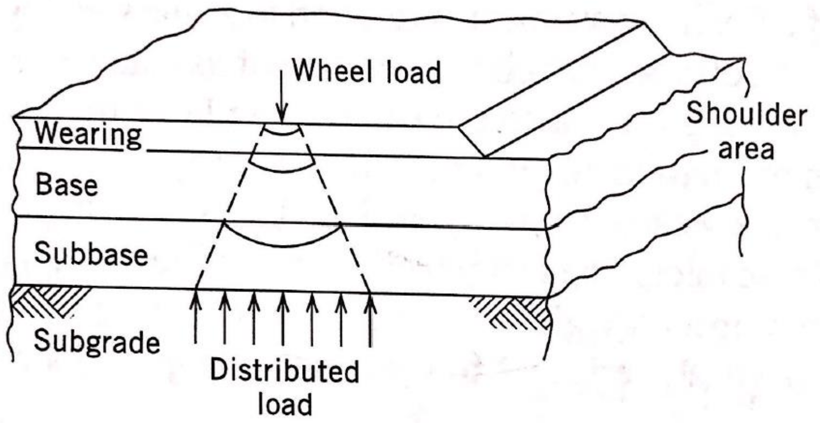


Figure 2.9: Distribution of load on a flexible pavement

2.6.1.3.1 Calculation of flexible pavement stresses and deflections

To design a pavement structure, one must be able to calculate the stresses and deflections in the pavement system. In the simplest case, the wheel load can be assumed to consist of a point load on a single-layer system as shown in Figure 2.10. This type of load and configuration can be analysed with Boussinesq solutions that were derived for soils analysis. The Boussinesq theory assumes that the pavement is one layer thick and the material is elastic, homogeneous and isotropic. The basic equation for the stress at a point in the system is

$$\sigma_z = K \frac{P}{z^2} \quad \text{(U.S. Customary)} \quad (2.10)$$

$$\sigma_z = 1000K \frac{P}{z^2} \quad \text{(Metric)} \quad (2.11)$$

Where;

σ_z = stress at point in kPa (Ib/in^2)
P = wheel load in N (Ib)
Z = depth of the point in question in mm (inches), and
K = variable defined as

$$K = \frac{3}{2\pi} \frac{1}{[1+ (r/z)^2]^{5/2}} \quad (2.12)$$

Where

r = radial distance in mm (inches) from the centreline of the point load to the point in question

Although the Boussinesq is useful for beginning the study of pavement stress calculations, it is not very representative of pavement system loading and configuration because it applies to a point load on one layer. A more realistic approach is to expand the point load to an elliptical area that represents a tire foot-print. The tire foot-print can be defined by an equivalent circular area with a radius calculated by

$$a = \sqrt{\frac{P}{p\pi}} \quad \text{……………………(U.S. Customary)………………………(2.13)}$$

$$a = \sqrt{\frac{P}{p\pi/1000}} \quad \text{……………………(Metric)…………………………..(2.14)}$$

Where:

a = equivalent load radius of the tire foot-print in mm (inches)

P = tire load in N (Ib)

p = tire pressure in kPa (Ib/in^2)

The integration of the load from a point to a circular area can be used to determine the stresses and deflections in a one-layer pavement system.

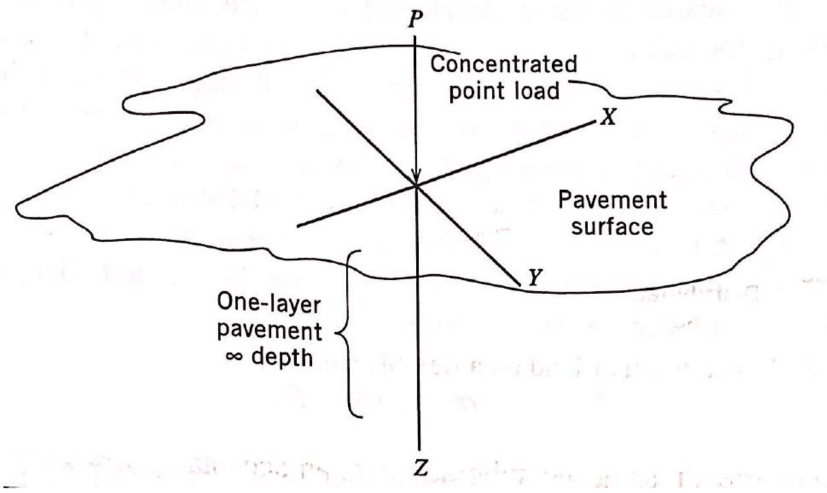


Figure 2.10: Point load on a one-layer pavement

However, Ahlivn and Ulery provided solutions for the evaluation of stresses, strain and deflections at any point in a homogenous half-space. Their work makes it easier to analyse a more complex pavement system than that considered in Boussinesq example. The one-layer equations by Ahlivin and Ulery can be used for material with any Poisson ratio which describes the change in width relative to length when a load is applied along the vertical axis. Based on Ahlvin and Ulery's work, the equation for the calculation of vertical stress is

$$\sigma_z = p(A + B) \quad \text{.......(2.15)}$$

The equation for radial-horizontal stress (which is a cause of pavement cracking) is

$$\sigma_r = p[2\mu A + C + (1 - 2\mu)F] \quad \text{.......(2.16)}$$

The equation for deflection is

$$\Delta_z = \frac{p(1+\mu)a}{E}\left[\frac{z}{a}A + (1-\mu)H\right] \quad \text{.......(2.17)}$$

Where:

σ_z = vertical stress in kPa (Ib/in^2)

σ_r = radial-horizontal stress in kPa (Ib/in^2)

Δ_z = deflection at depth z in mm (inches)

p = pressure due to the tire load in kPa (Ib/in^2)

μ = Poisson ratio

E = modulus of elasticity (known as Young's modulus, the ratio of stress to strain as a load is applied to a material in kPa (Ib/in^2) and

A,B,C,F and H = function values as presented in Table 2.3 that depends on z/a and r/a, the depth in radii and offset distance in radii respectively

Where

z = depth of the point in question in mm (inches)

r = radial distance in mm (inches) from the centreline of the point load to the point in question

a = equivalent load radius of the tire foot-print in mm (inches)

Example 2.1: *A tire with 689 kPa air pressure distributes a load over an area with a circular contact radius, a, of 127mm. The pavement was constructed with a material that has a modules of elasticity of 345000 kPa and a Poisson ratio of 0.45. Calculate the radial-horizontal stress and deflection at a point the pavement surface under the centre of the ire load. Also, calculate the radial-horizontal stress and deflection at a point at a depth of 508mm and radial distance of 254mm from the centre of the tire load. (USE: Ahlvin and Ulery equations).*

2.6.1.3.1 Design of thickness layers based on AASHTO 1993

There are several accepted flexible pavement design procedures, including the Asphalt Institute method, the National Stone Association procedure, and the Shell procedure. Most of the procedures have been field verified and used by highway agencies for several years. The selection of one procedure over another is usually based on a highway agency's experience and satisfaction with design results.

A widely accepted flexible pavement design procedure is presented in the AASHTO Guide for design of pavement structures, which is published by the American Association of State Highway and Transportation Officials. This design method is based on AASHO test results conducted in Illinois, USA. The procedure was first published in 1972, with latest revision in 1993. The factors considered in this design methods are as follows:

- *Pavement performance*
- *Traffic*
- *Roadbed/Subgrade soils*
- *Construction materials*
- *Environmental factors*
- *Drainage*
- *Reliability*

Pavement performance

There are two factors considered under the performance of the pavement structure, these are

- ✓ Structural performance: this is related to the physical factors that affect load-carrying capacity.

- ✓ Functional performance: this is related to factors that affect the riding quality.

A serviceability performance concept was used for pavement performance quantification. Under this concept, a present serviceability index (PSI) was developed which range from 1 to 5 (typical condition after pavement construction)

For the purpose of pavement design procedure, two serviceability indices are used:

- ❖ Initial serviceability index (pi): serviceability index immediately after pavement construction.
- ❖ Final serviceability index (pt): serviceability index at which pavement needs maintenance.

pi: 4.2 for flexible pavement

pt:

- 2.5 – 3 for major highways
- 2 for lower classification
- 1.5 for extreme economic conditions (limited fund)

ΔPSI = pi - pt

<u>Subgrade soils condition</u>

In AASHTO 1993 procedure, the resilient modulus (Mr) is used to represent subgrade property. However, due to the availability and cheapness of the CBR test as compared with resilient modulus test, the following formula are used

Mr = 1500 x CBR

Materials of construction

These can be classified into:

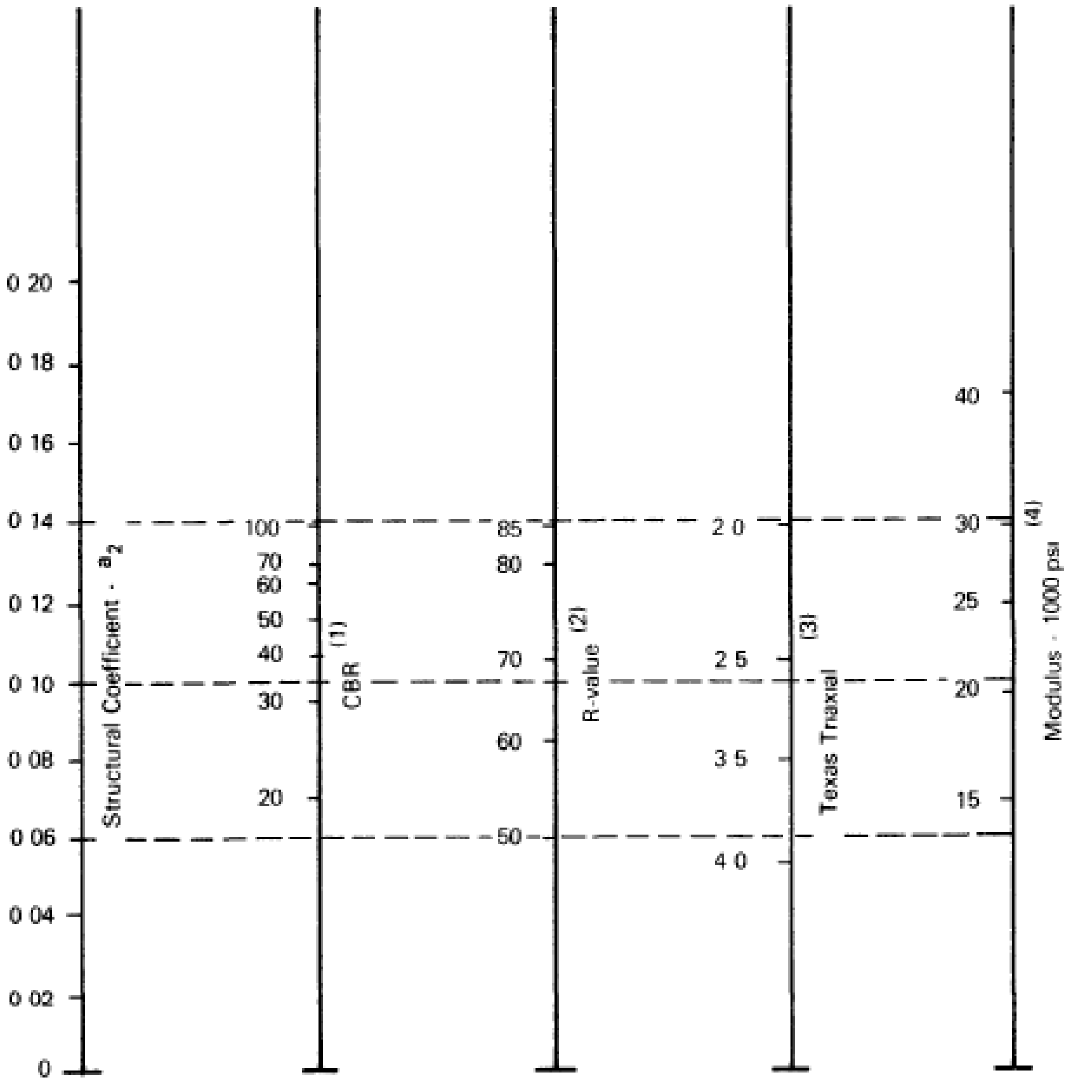
1. Materials for subbase course.

2. Materials for base course.

3. Materials for surface/wearing course

- ***Materials for subbase course***

Layer coefficient is used in pavement design procedure to reflect the layer quality and used to convert the layer thickness to the structural number (SN). Figure 2.11 is used for granular subbase to convert different material properties to equivalent layer coefficient (a3).



(1) Scale derived by averaging correlations obtained from Illinois
(2) Scale derived by averaging correlations obtained from California, New Mexico and Wyoming
(3) Scale derived by averaging correlations obtained from Texas
(4) Scale derived on NCHRP project *(3)*

Figure 2.12: Estimation of base layer coefficient a2

- ***Materials for base course***

Materials used for base course should satisfy the general requirements such as gradation and other requirements. Figures 2.12, 2.13 and 2.14 are used for granular, cement-treated and bituminous treated base layers, respectively, to convert layer properties to the structural layer coefficient (a2).

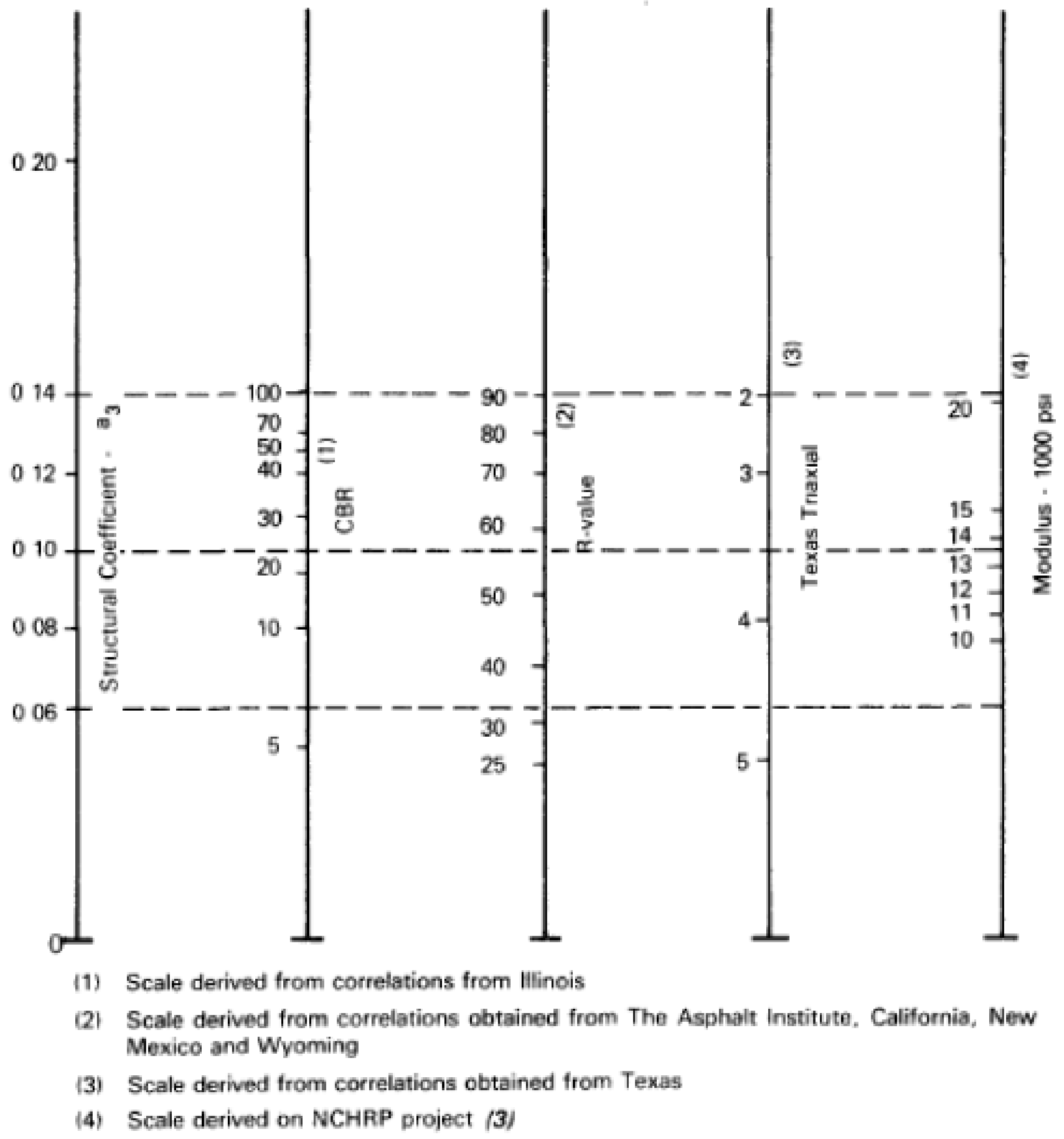
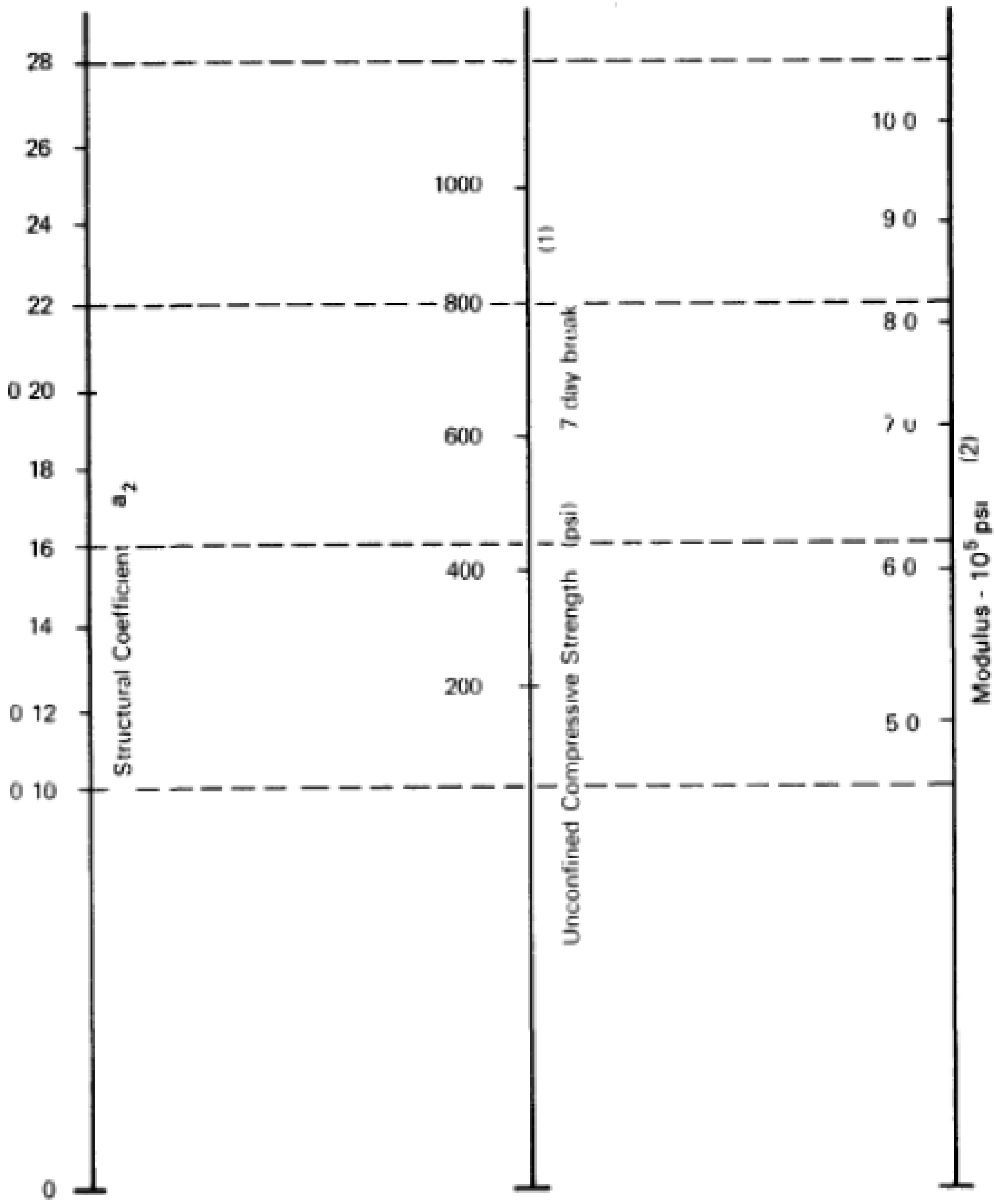


Figure 2.11: Estimation of subbase layer coefficient a3



(1) Scale derived by averaging correlations from Illinois Louisiana and Texas

(2) Scale derived on NCHRP project (3)

Figure 2.13: Estimation of base layer coefficient a2- cement treated base course

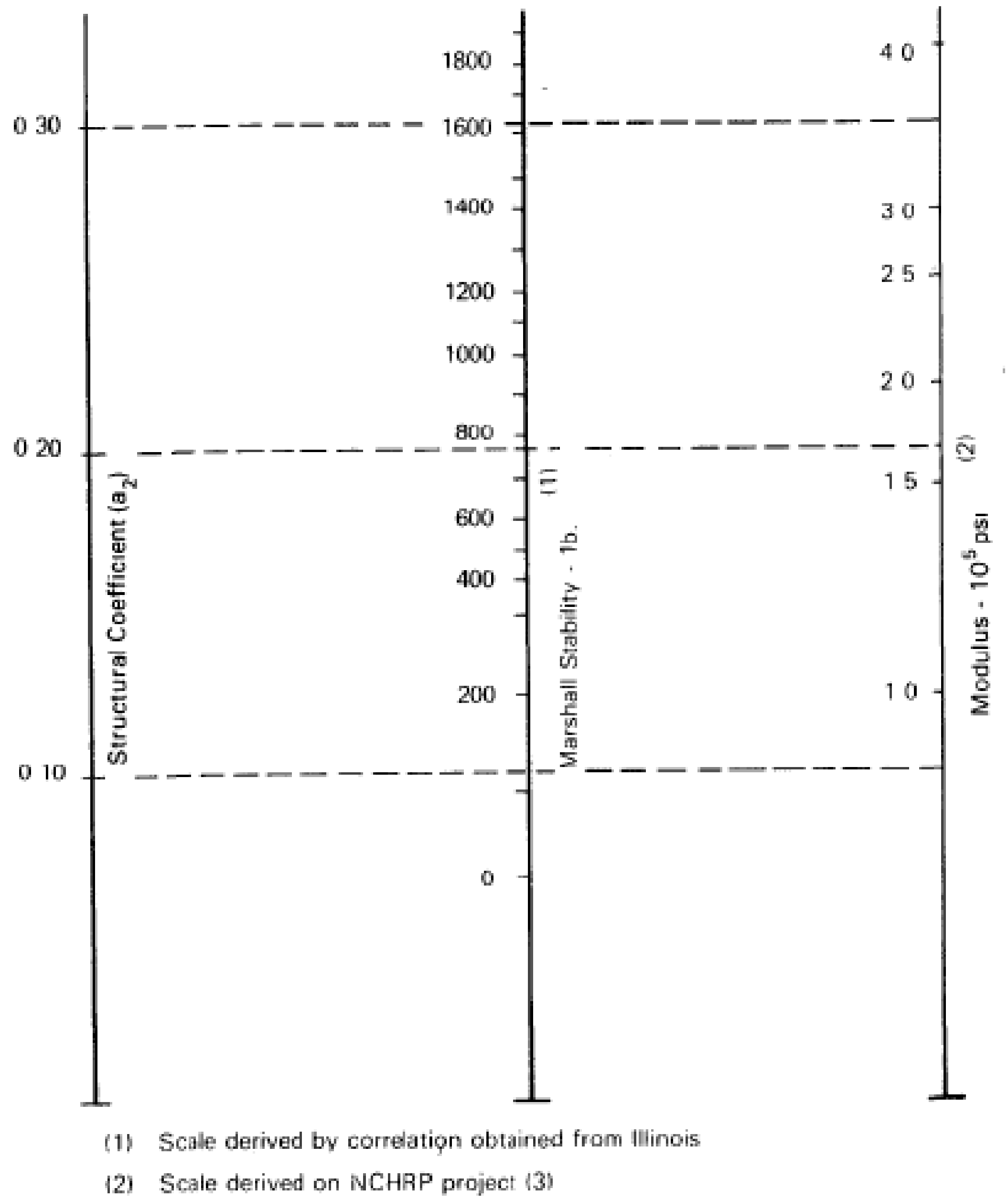


Figure 2.14: Estimation of base layer coefficient a2-bitumenous treated base course

- ***Materials for surface course***

Hot Mix Asphalt (HMA) mixtures are frequently used for surface course construction. Dense-graded mixtures are normally used for such purposes. Figure 2.15 was suggested to estimate layer coefficient (a1) for HMA.

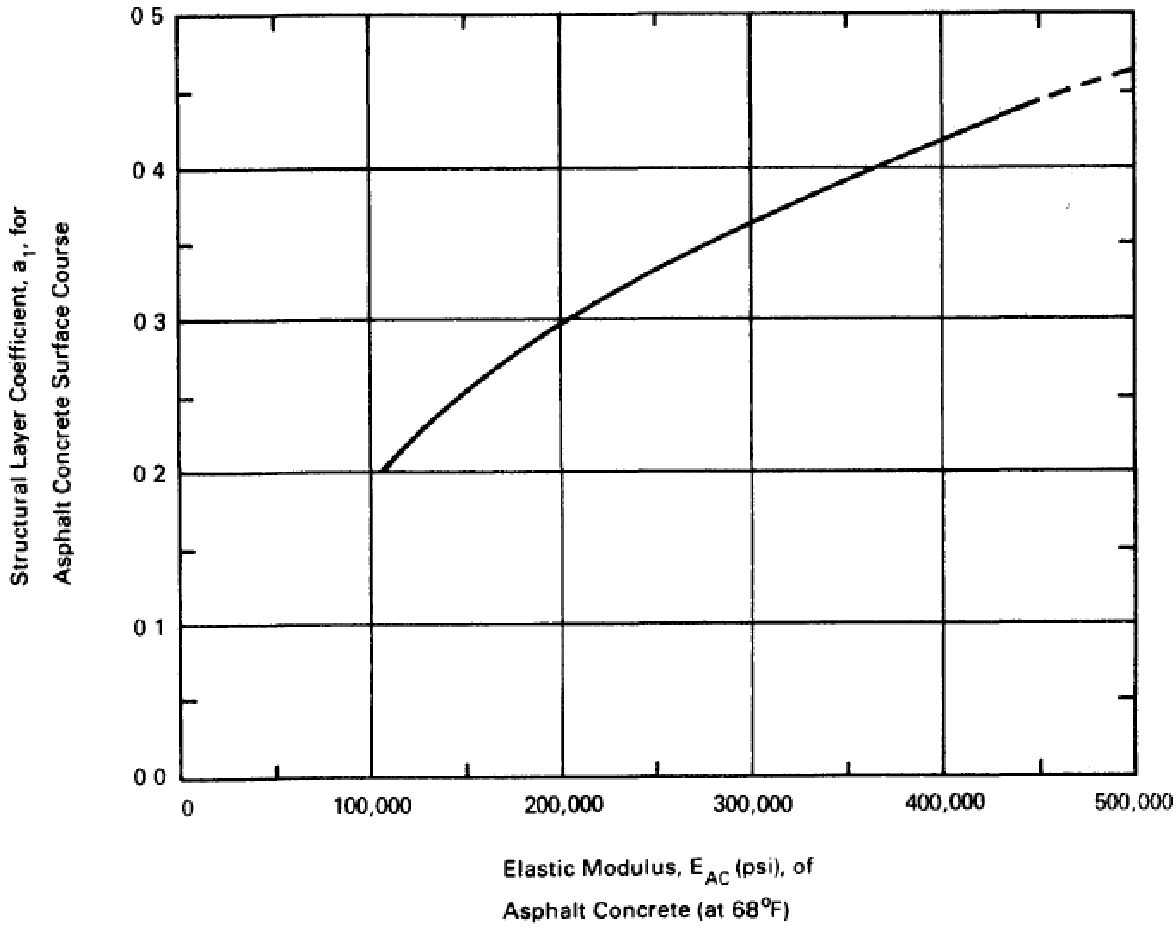


Figure 2.15: Estimation of surface layer coefficient a1

Environmental factors

As discussed previously the main two environmental factors taken onto consideration are temperature and rainfall Figure 2.16 shows pavement performance trends. Temperature affects the thermal properties of pavement materials and also freeze-thaw of the subgrade soil. Rainfall also affects the performance of the pavement and roadbed soil especially when it penetrates into the underlying layers. The results found that the subgrade soil properties are changing during the year due to increase the temperature and thaw period.

AASHTO procedure takes these seasonal variations during the year into consideration through determination of the effective subgrade resilient modulus (Mr effective.) using the following procedure:

- The whole year is divided into 12 periods and the resilient modulus should be measured for each of these periods.
- The relative damage should be estimated for each period from the following formula

  $$uf = 1.18 \times 10^{8} \times Mr^{-2.32}$$

- The average relative damage is calculated then; the effective resilient modulus can be determined using the same formula above

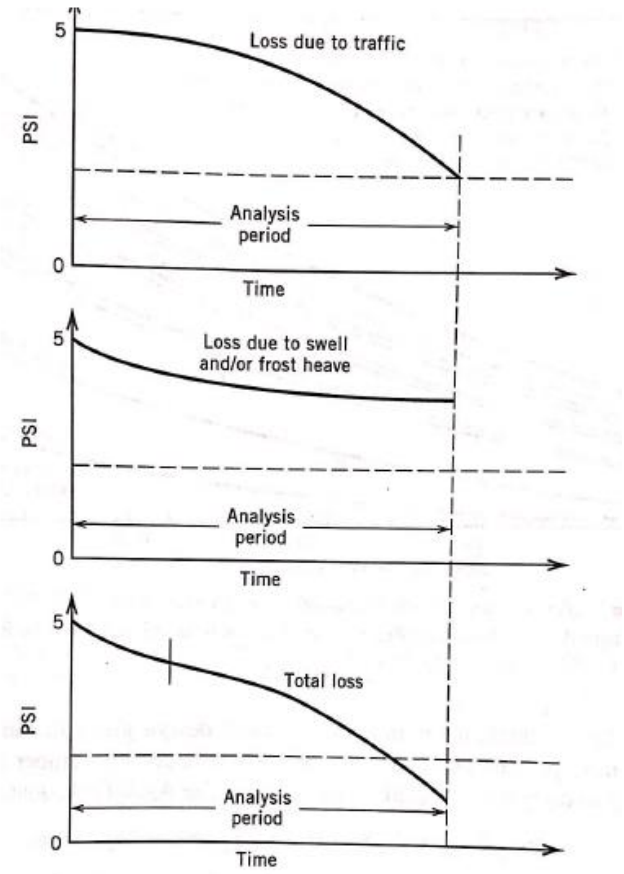


Figure 2.16: Pavement performance trend

Example 2.2: Table below (column 1 and column 2) shows the results of laboratory testing for the resilient modulus of subgrade soil during 12 months. Estimate the effective resilient modulus for this subgrade for pavement design?

$uf = 1.18 * 10^{8} * Mr^{-2.32}$

Month	*Subgrade resilient modulus (psi)*	*Damage (uf)*
Jan.	*22000*	*0.01*
Feb.	*22000*	*0.01*
Mar.	*5500*	*0.25*
Apr.	*5000*	*0.3*
May	*5000*	*0.3*
Jun.	*8000*	*0.1*
Jul.	*8000*	*0.1*
Aug.	*8000*	*0.1*
Sep.	*8500*	*0.09*
Oct	*8500*	*0.09*
Nov.	*6000*	*0.2*
Dec.	*22000*	*0.01*
Summation uf		*1.59*

Average uf =0.133 then ***Mr eff. = <u>7250 psi</u>***

Drainage

AASHTO 1993 guide considers the presence of water within granular base or subbase courses affect their strength. Consequently, they suggested a drainage coefficient to modify the structural performance of the mentioned layers. This can be conducted by incorporating drainage factors (mi) for base and subbase layers. These factors depend both on quality of drainage and the percentages of time during which the pavements structure is saturated. The quality of drainage is measured in terms of the time required to reduce the degree of saturation to

50%. Tables 2.4 is used to estimate the quality of drainage while Table 2.5 is for drainage factors estimation.

Table 2.4: Estimation of drainage quality

Quality of Drainage	Water Removed Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very poor	(water will not drain)

Table 2.5: Drainage coefficient for modifying structural coefficients for untreated bases and subbases

	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
Quality of Drainage	**Less Than 1%**	**1–5%**	**5–25%**	**Greater Than 25%**
Excellent	1 40–1 35	1 35–1 30	1 30–1 20	1 20
Good	1 35–1 25	1 25–1 15	1 15–1 00	1 00
Fair	1 25–1 15	1 15–1 05	1 00–0 80	0 80
Poor	1 15–1 05	1 05–0 80	0 80–0 60	0 60
Very poor	1 05–0 95	0 95–0 75	0 75–0 40	0 40

Reliability

Due the importance that the traffic loads estimation has on the pavement design process, reliability issue was introduced to take into consideration the uncertainty of the traffic loads estimation. This was done by incorporating reliability factors (F_R) which depends on two factors. One of these is the reliability design level (R) which is the level of assurance that the pavement section designed will survive for the whole design period. Table 2.6 shows suggested reliability levels for different highway types. The other factor is the overall variation S_o^2 which account for the variation in the traffic forecasts and actual pavement performance. Reliability level is computed based on the following expression

Z_R = standard normal variation for a given reliability level.

S_o = overall standard deviation.

Table 2.7 presents the Z_R values for various reliability levels while Table 2.8 shows the overall standard deviation S_o for both flexible and rigid pavements.

Table 2.6: Suggested level of Reliability for various functional classifications

	Recommended Level of Reliability	
Functional Classification	*Urban*	*Rural*
Interstate and other freeways	85–99.9	80–99.9
Other principal arterials	80–99	75–95
Collectors	80–95	75–95
Local	50–80	50–80

Note: Results based on a survey of the AASHTO Pavement Design Task Force.
SOURCE: Adapted with permission from *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C., 1993.

Table 2.7: Standard normal deviation (Z_R) values corresponding to select level of Reliability

Reliability (R%)	*Standard Normal Deviation, Z_R*
50	−0.000
60	−0.253
70	−0.524
75	−0.674
80	−0.841
85	−1.037
90	−1.282
91	−1.340
92	−1.405
93	−1.476
94	−1.555
95	−1.645
96	−1.751
97	−1.881
98	−2.054
99	−2.327
99.9	−3.090
99.99	−3.750

SOURCE: Adapted with permission from *AASHTO Guide for Design of Pavement Structures*, American

Table 2.8: S_o values for different pavements

	Standard Deviation, S_o
Flexible pavements	0.40–0.50
Rigid pavements	0.30–0.40

Structural design

The objective of the AASHTO 1993 pavement design method is to determine the Structural Number (SN) adequate to withstand the design traffic loads (in terms of ESAL) for design period. It should be noted that the current design procedure is used for ESAL value above 50,000. Roads those carrying less than this values are classified as low volume roads. Structural number can be computed from the following equation

$SN = a_1 . D_1 + a_2 . m_2 . D_2 + a_3 . m_3 . D_3$

where

a_1, a_2, a_3 = structural layer coefficients

m_2, m_3 = drainage factors.

D_1, D_2, D_3 = surface, base and subbase thicknesses

AASHTO 1993 use the following design equation to perform the structural design

$$\log_{10} W_{18} = Z_R S_o + 9.36 \log_{10}(SN + 1) - 0.20 + \frac{\log_{10}[\Delta PSI/(4.2 - 1.5)]}{0.40 + [1094/(SN + 1)^{5.19}]} + 2.32 \log_{10} M_r - 8.07$$

Where:

W_{18} = predicted number of 18,000-lb (80 kN) single-axle load applications
Z_R = standard normal deviation for a given reliability
S_o = overall standard deviation
SN = structural number indicative of the total pavement thickness
$\Delta PSI = p_i - p_t$

p_i = initial serviceability index
p_t = terminal serviceability index
M_r = resilient modulus (lb/in^2)

Figure 2.17 is a solution of equation above and used for SN estimation.

Minimum layers' thicknesses

In terms of practicality and economics considerations, AASHTO suggest that the layer thickness should not be less than the values presented in Table 2.9.

Table 2.9: AASHTO-Recommended minimum thicknesses for highway layers

	Minimum Thickness (in.)	
Traffic, ESALs	*Asphalt Concrete*	*Aggregate Base*
Less than 50,000	1.0 (or surface treatment)	4
50,001–150,000	2.0	4
150,001–500,000	2.5	4
500,001–2,000,000	3.0	6
2,000,001–7,000,000	3.5	6
Greater than 7,000,000	4.0	6

SOURCE: Adapted with permission from *AASTHO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C., 1993.

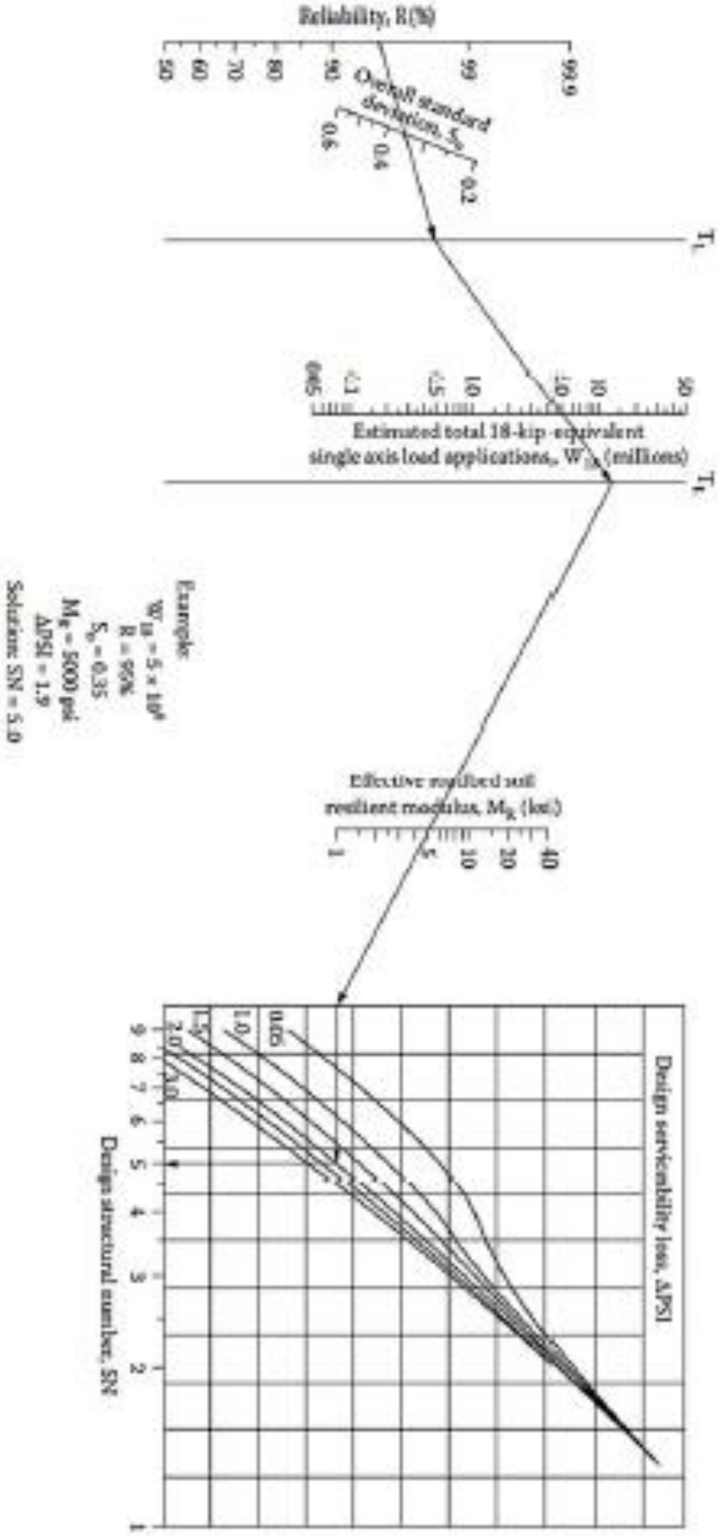


Figure 2.17: Nomograph for thickness design estimation

Example 2.3: A flexible pavement is to be designed to carry ESAL of 2 x 10^6 applied on an urban interstate highway. It is required about a week to drain the water from within the pavement structures. It is estimated that the pavement structure will be saturated for 30% of the time. The other information as resulted from experimental design is as follows

Resilient modulus of the asphalt concrete at 68° F = 450,000 psi

CBR value of base course = 100, Mr = 31,000 psi

CBR value of subbase course = 22, Mr = 13,500 psi

CBR value of subgrade materials = 6

Initial serviceability index Pi=4.5

Terminal serviceability index pt=2.5

Design a suitable pavement structure in accordance with AASHTO 1993 method?

Sol.

Reliability Level R = 99% (Table 2.6)

Standard deviation $\boldsymbol{S_o}$ **=** 0.49 (Table 2.8)

Use the nomograph illustrated in Figure 2.17 to design the pavement as follows:

Step 1: Draw a line joining the reliability level of 99% and the overall standard deviation S_o of 0.49, and extend this line to intersect the first T_L line at point A.

Step 2: Draw a line joining point A to the ESAL of 2 x 10^6, and extend this line to intersect the second T_L line at point B.

Step 3: Draw a line joining point B and resilient modulus (M_r) of the roadbed soil, and extend this line to intersect the design serviceability loss chart at point C

Step 4: Draw a horizontal line from point C to intersect the design serviceability loss (**ΔPSI)** curve at point D. In this example, **ΔPSI = 4.5-2.5 =2**

Step 5: Draw a vertical line to intersect the design SN, and this value SN= 4.4

Step 6: Determine the appropriate structure layer coefficient for construction materials

1- Resilient value of asphalt = 450000 Ib/in^2. From Figure 2.15, a_1 = 0.44
2- CBR of base course material = 100. From Figure 2.12, a_2= 0.14
3- CBR of subbase course material = 22. From Figure 2.11, a_3= 0.1

Step 7: Determine appropriate drainage coefficient mi. since only one set of conditions is given for both the base and subbase layers, the same value will be used for m_1 and m_2. The time required for water to drain from within pavement = 1 week and from Table 2.4, drainage quality is fair. The percentage of time pavement structure will be exposed to moisture levels approaching saturation = 30 and from Table 2.5, m_i = 0.8

Step 8: Determine appropriate layer thickness from the following equation

$= a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3$

Taking into consideration that a flexible pavement structure is a layered system, the determination of the different thickness should be carried out as indicated in Figure 2.18. The required SN above the subgrade is first determined, and then the required SN above the base and subbase layers are determined using the appropriate strength of each layer. The minimum allowable thickness of each layer can then be determine using the differences of the computed SNs as shown in Figure 2.18

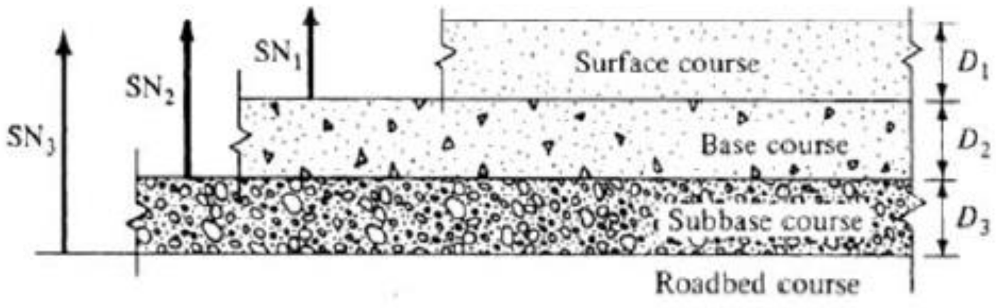


Figure 2.18: Procedure for determining thickness of layers using a layered analysis

Using the appropriate values for Mr in figure 2.18, we obtain SN_3 = 4.4 and SN_2 = 3.8. Note that when SN is assumed to compute ESAL, the assumed and computed SN_3 must be approximately equal. If these are significantly different, the computation must be repeated with a new assumed SN.

Mr for base course = 31000 Ib/in^2

Using this value in Figure 2.17, we obtain

$SN_1 = 2.6$

Giving

$D_1 = \frac{2.6}{0.44} = 5.9$ in

Using 6 in for the thickness of surface course,

$D^*_1 = 6$ in

$SN^*_1 = a_1 D^*_1 = 0.44 * 6 = 2.64$

$D^*_2 \geq \frac{SN_2 - SN_1^*}{a_2 m_2} \geq \frac{3.8-2.64}{0.14*0.8} \geq 10.36$ in (use 12 in)

$SN^*_2 = 0.14 * 0.8 * 12 + 2.64 = 1.34 + 2.64$

$D^*_3 \geq \frac{SN_3 - SN_2^*}{a_3 m_3} \geq \frac{4.4-(1.34+2.64)}{0.1*0.8} \geq 5.25$ in (use 6 in)

SN^*_3 = 2.64+ 1.34+ 0.1*0.8*6 = 4.46

The pavement will therefore consist of 6 in asphalt concrete surface, 12 in granular base, and 6 in subbase.

- * with D or SN indicates that it represents the value actually used which must be equal to or greater than the required value

2.6.2 Rigid Pavement

This type of pavement consists of concrete slab and base course (when used in rigid pavement it is called subbase course). These two layers are resting on subgrade soil.

2.6.2.1 Rigid Pavement Types

1. Jointed plain concrete pavement (JPCP)

2. Jointed reinforced concrete pavement (JRCP)

3. Continuous reinforced concrete pavement (CRCP)

4. Prestressed concrete pavements

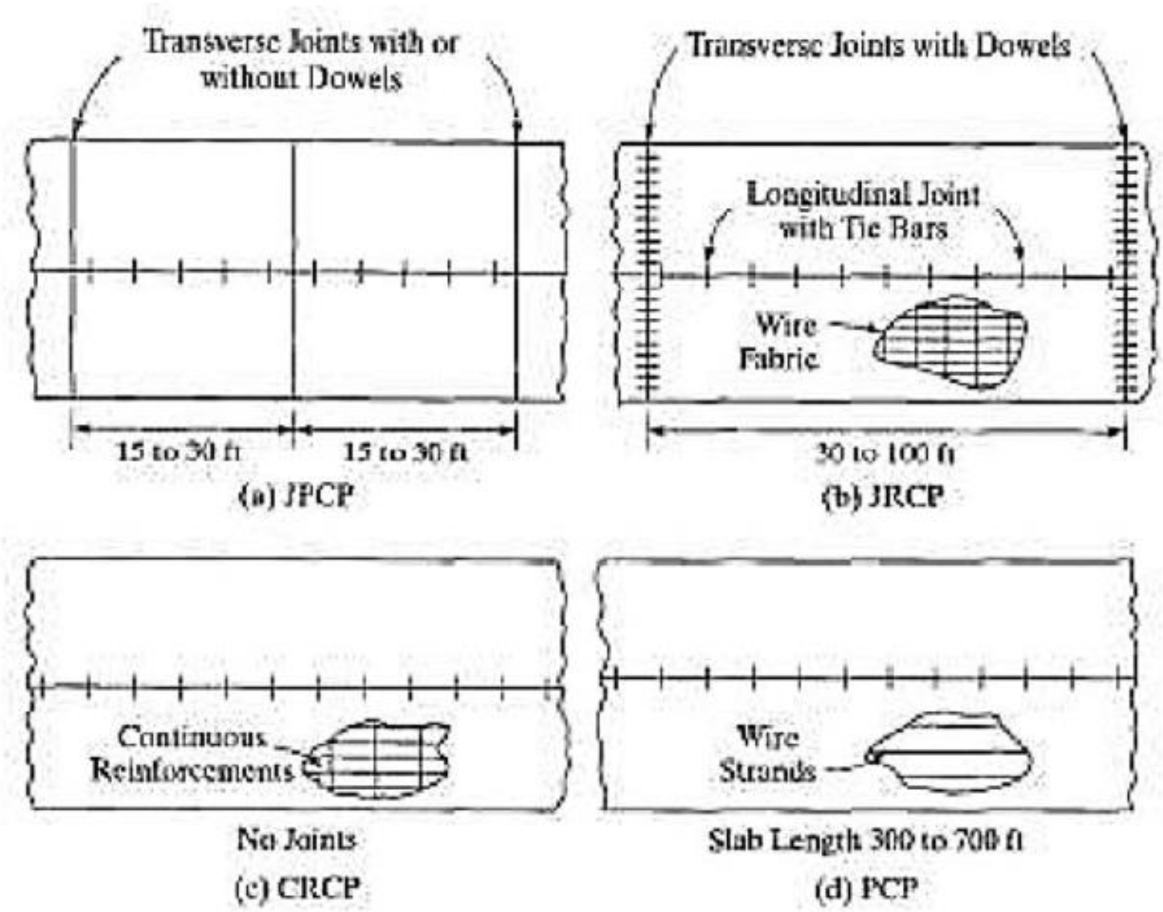


Figure 2.18: Different concrete pavement types (plan views)

2.6.2.2 Rigid Pavement Reinforcements

1. Temperature reinforcement (wire fabric)

2. Tie bars (1- Prevent lanes from separation and differential deflection. 2- Reduce transverse cracking)

3. Dowel bars (1- Minimise deflections and reduce stresses near the edges of the slabs. 2- Transferee load from one slab to another without preventing the joint from opening)

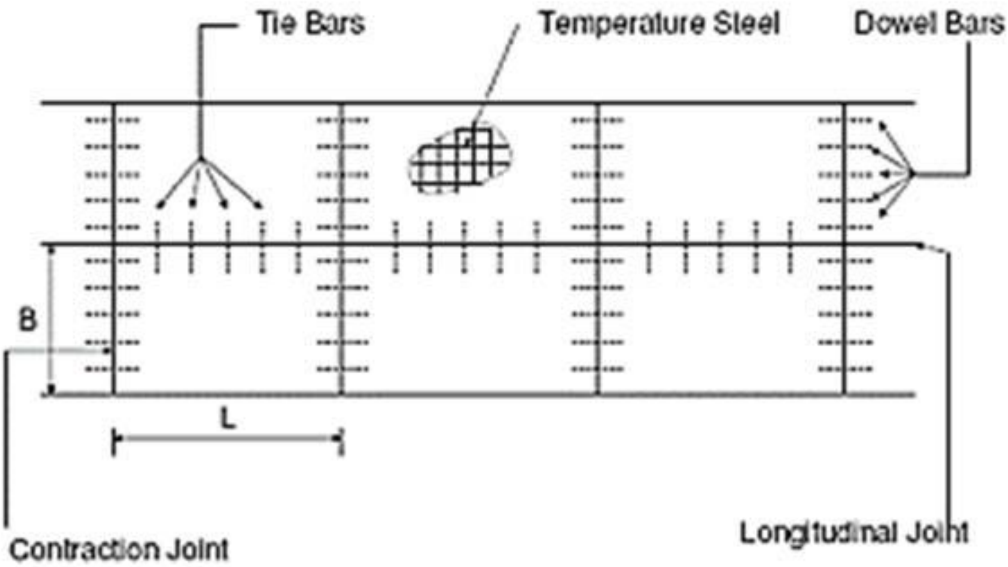


Figure 2.19: **Reinforcement illustration and general appearance of concrete pavement**

2.6.2.3 Joints in rigid pavement

1. Expansion joints

It is placed at a specific location to allow the pavement to expand without damaging adjacent structures or the pavement itself as shown in Figure 2.20

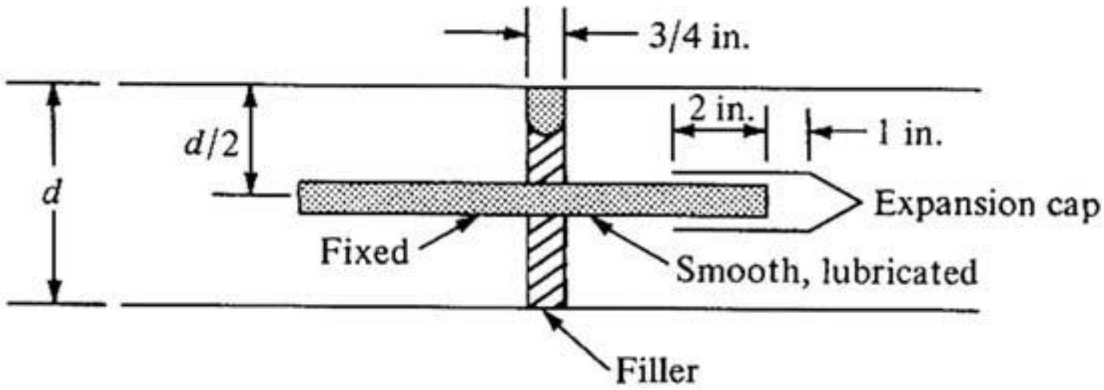


Figure 2.20: **Expansion joint**

2. Contraction joint

is a sawed, formed, or tooled groove in a concrete slab that creates a weakened vertical plane. It regulates the location of the cracking caused by dimensional changes in the slab.

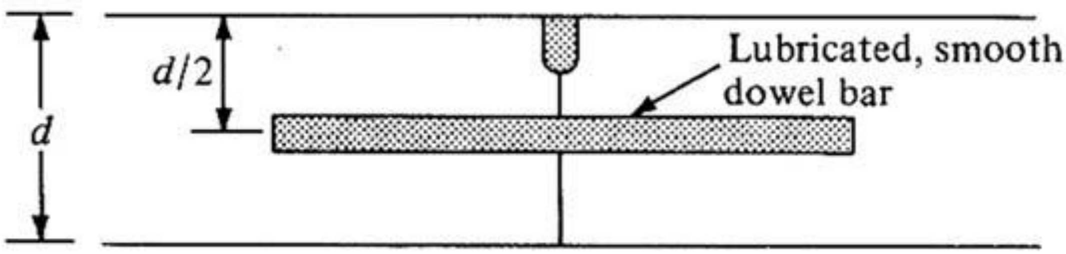


Figure 2.21: **Contraction joint**

3. Construction joints

It is a joint between slabs that results when concrete is placed at different times. Figure 2.22 shows this type of joint.

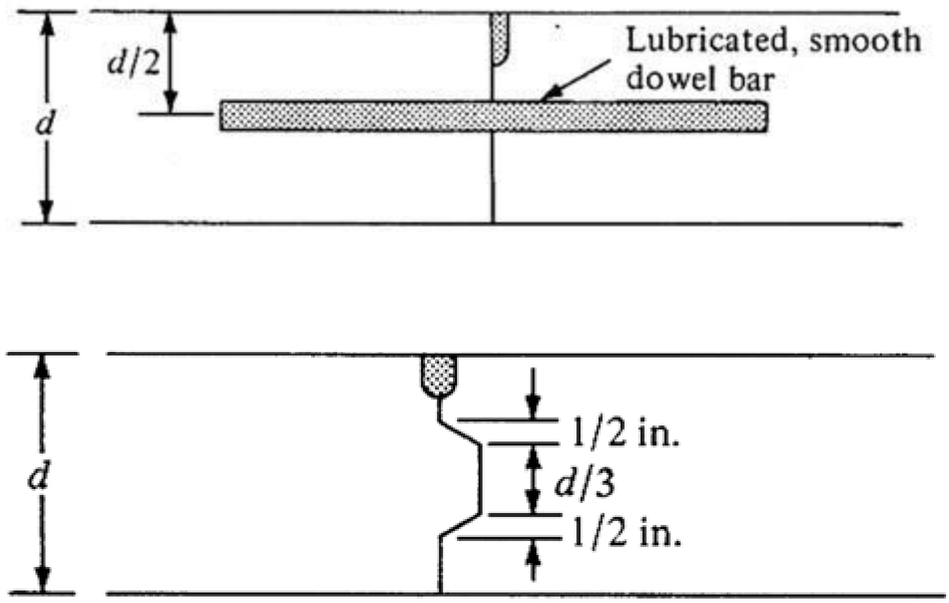


Figure 2.22: **Construction joint**

2.6.2.4 AASHTO 1993 thickness design method

In design of rigid pavement, AASHTO design method consider many factors. These are:

1. Pavement performance.

2. Subbase strength.

3. Subgrade strength.

4. Traffic.

5. Concrete properties.

6. Drainage.

7. Reliability.

Pavement performance

This factor is considered in similar way of that discussed in flexible pavement. However, the initial serviceability index (pi) is taken as 4.5. Final serviceability index value is as discussed in flexible pavement design.

Subbase strength

AASHTO 1993 design guide allow using of six subbase types (A to F) ranging from granular to stabilized materials. The requirements needed for these types are shown in Table 2.10. The minimum thickness as suggested by AASHTO method should not be less than 6 inches and this should be extended 1-3 ft outside pavement edges. Subbase materials is characterized by its elastic modulus E_{SB}.

Table 2.10: Recommended particle size distributions for different types of subbase materials

	Types of Subbase					
Sieve Designation	*Type A*	*Type B*	*Type C (Cement Treated)*	*Type D (Lime Treated)*	*Type E (Bituminous Treated)*	*Type F (Granular)*
Sieve analysis percent passing						
2 in.	100	100	—	—	—	—
1 in.	—	75–95	100	100	100	100
⅜ in.	30–65	40–75	50–85	60–100	—	—
No. 4	25–55	30–60	35–65	50–85	55–100	70–100
No. 10	15–40	20–45	25–50	40–70	40–100	55–100
No. 40	8–20	15–30	15–30	25–45	20–50	30–70
No. 200	2–8	5–20	5–15	5–20	6–20	8–25
(The minus No. 200 material should be held to a practical minimum.)						
Compressive strength lb/in^2 at 28 days			400–750	100		
Stability						
Hveem Stabilometer					20 min	
Hubbard field					1000 min	
Marshall stability					500 min	
Marshall flow					20 max	
Soil constants						
Liquid limit	25 max	25 max				25 max
Plasticity index[a]	N.P.	6 max	10 max[b]		6 max[b]	6 max

[a]As performed on samples prepared in accordance with AASHTO Designation T87.
[b]These values apply to the mineral aggregate prior to mixing with the stabilizing agent.
SOURCE: Adapted with permission from *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, American Association of State Highway and Transportation Officials, Washington, D.C., 2007.

Subgrade strength

To reflect the property of subgrade soils in rigid pavement design procedure, the modulus of subgrade reaction (k) is normally used. This modulus can be estimated by conducting plate bearing test. However, the correlation with other tests is used sometime to estimate this modulus. Figure 2.23 is an example of such correlations.

Current pavement design procedure suggests the use of effective modulus of subgrade reaction. This effective modulus depends on the following factors

1. Seasonal effect on subgrade resilient modulus (as discussed and illustrated in flexible pavement)

2. The type and the thickness of subbase layer being used.

3. The effect of subbase potential erosion.

4. The presence of bedrock layer within the 10 foot below the subgrade surface.

The composite modulus of subgrade reaction can be determined from Figure 2.24. The effective modulus of subgrade reaction is then computed to account for the potential erosion of subbase course. This can be done using Table 2.11 and Figure 2.25.

Furthermore, as mentioned previously, the presence of a bedrock layer within 10 foot below surface of subgrade may affect the subgrade reaction modulus. In fact presence of such rigid foundation may have a positive effect on the overall modulus. Figure 2.26 is used to take such effect into account.

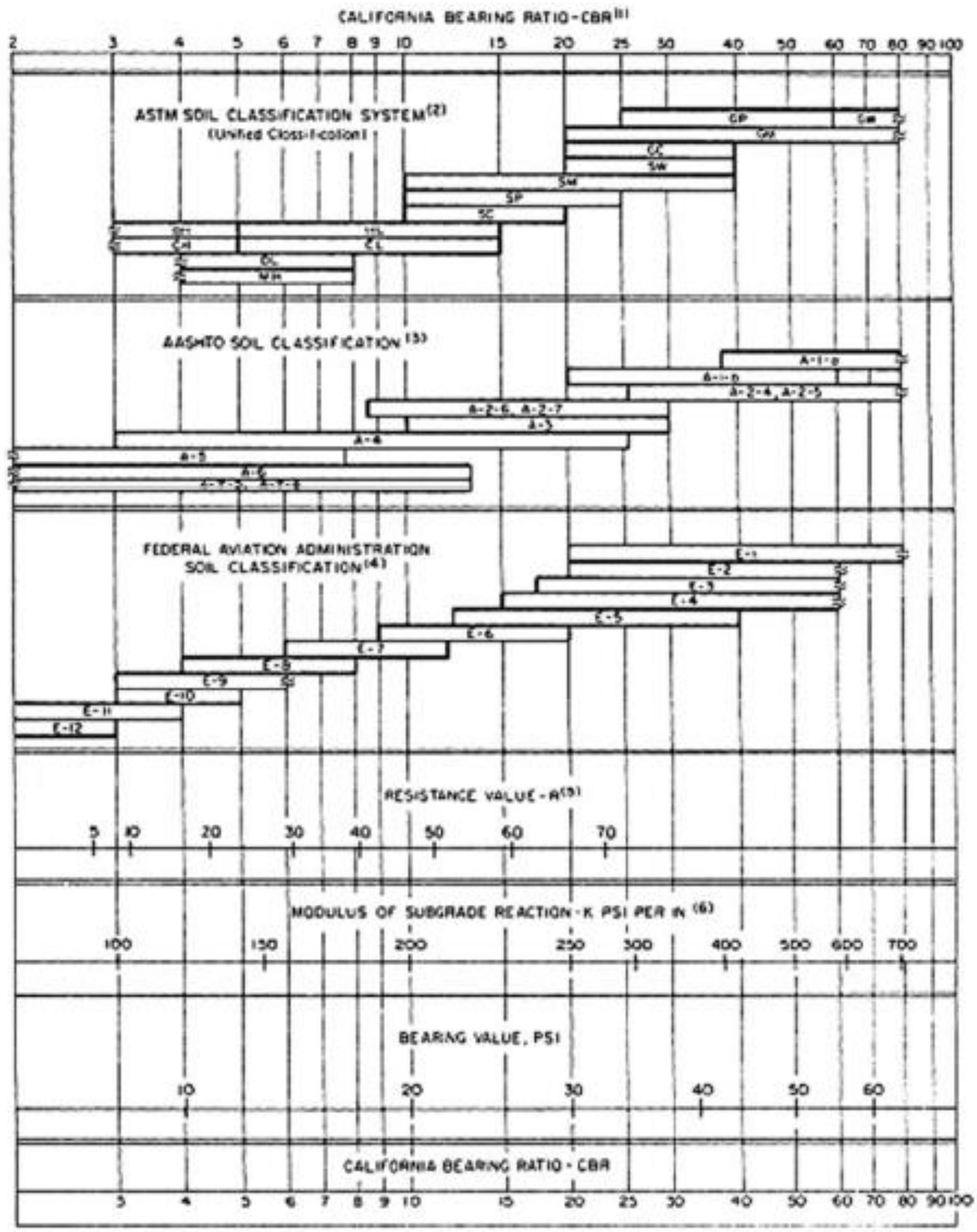
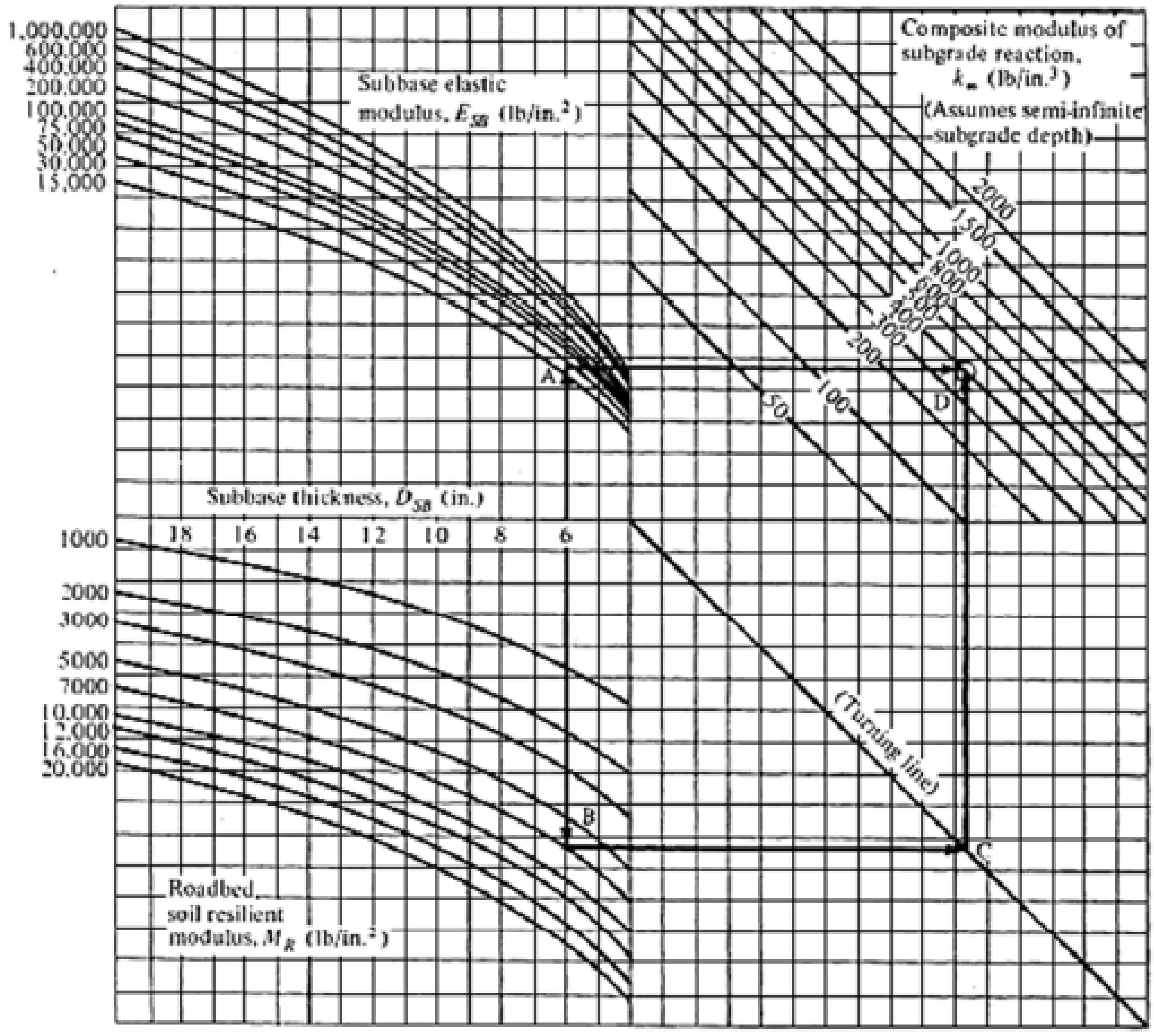


Figure 2.23: Correlation of different soils properties



Example:

D_{SB} = 6 in.

E_{SB} = 20,000 lb/in.^{2}

M_R = 7,000 lb/in.^{2}

Solution: k_∞ = 400 lb/in.^{3}

Figure 2.24: Chart for estimation the composite modulus of subgrade reaction

Table 2.11: Loss of support (LS) factors

Typical Ranges of Loss of Support (LS) Factors for Various Types of Materials (*6*)

Type of Material	Loss of Support (LS)
Cement Treated Granular Base (E = 1,000,000 to 2,000,000 psi)	0 0 to 1 0
Cement Aggregate Mixtures (E = 500,000 to 1,000,000 psi)	0 0 to 1 0
Asphalt Treated Base (E = 350,000 to 1,000,000 psi)	0 0 to 1 0
Bituminous Stabilized Mixtures (E = 40,000 to 300,000 psi)	0 0 to 1 0
Lime Stabilized (E = 20,000 to 70,000 psi)	1 0 to 3 0
Unbound Granular Materials (E = 15,000 to 45,000 psi)	1 0 to 3 0
Fine Grained or Natural Subgrade Materials (E = 3,000 to 40,000 psi)	2 0 to 3 0

NOTE: E in this table refers to the general symbol for elastic or resilient modulus of the material

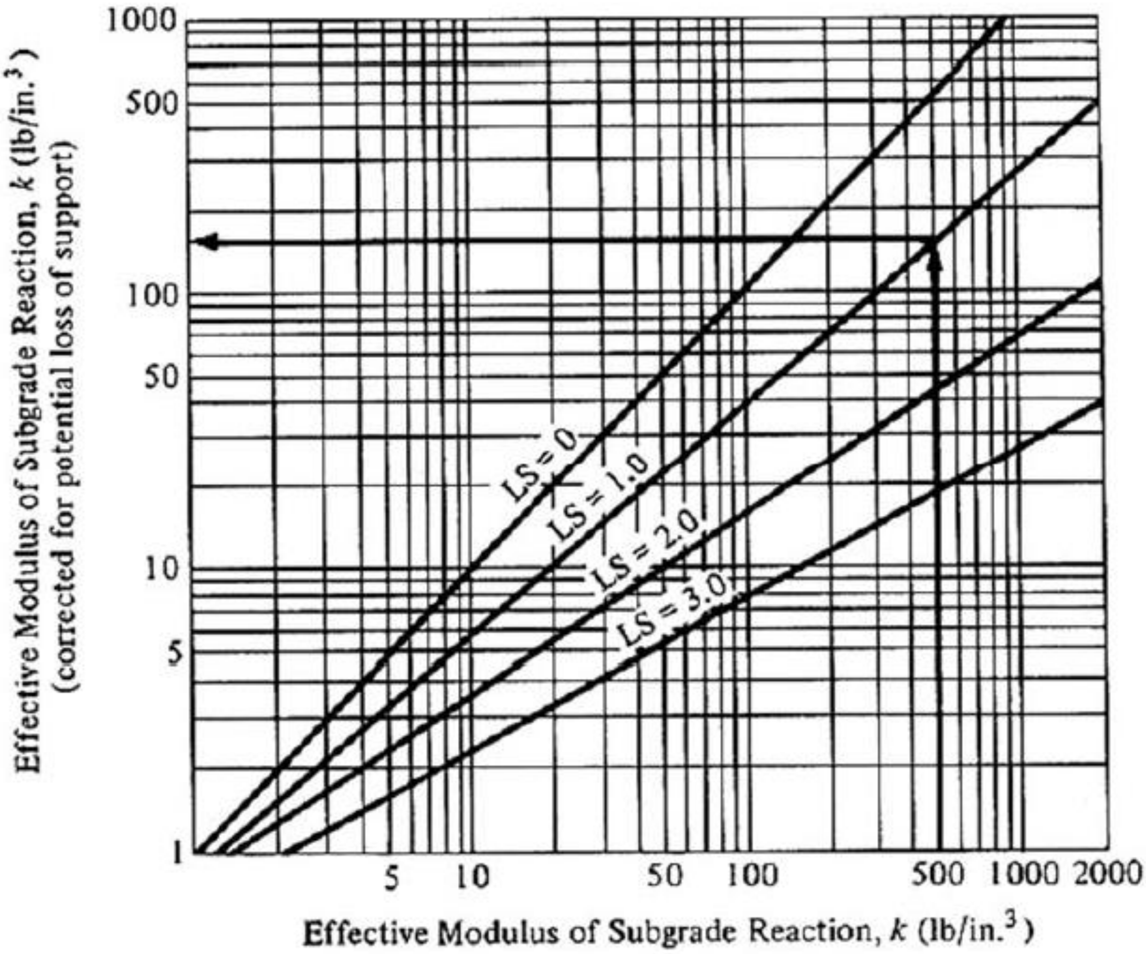


Figure 2.25: Chart for estimation the effective modulus of subgrade reaction corrected for potential loss of support

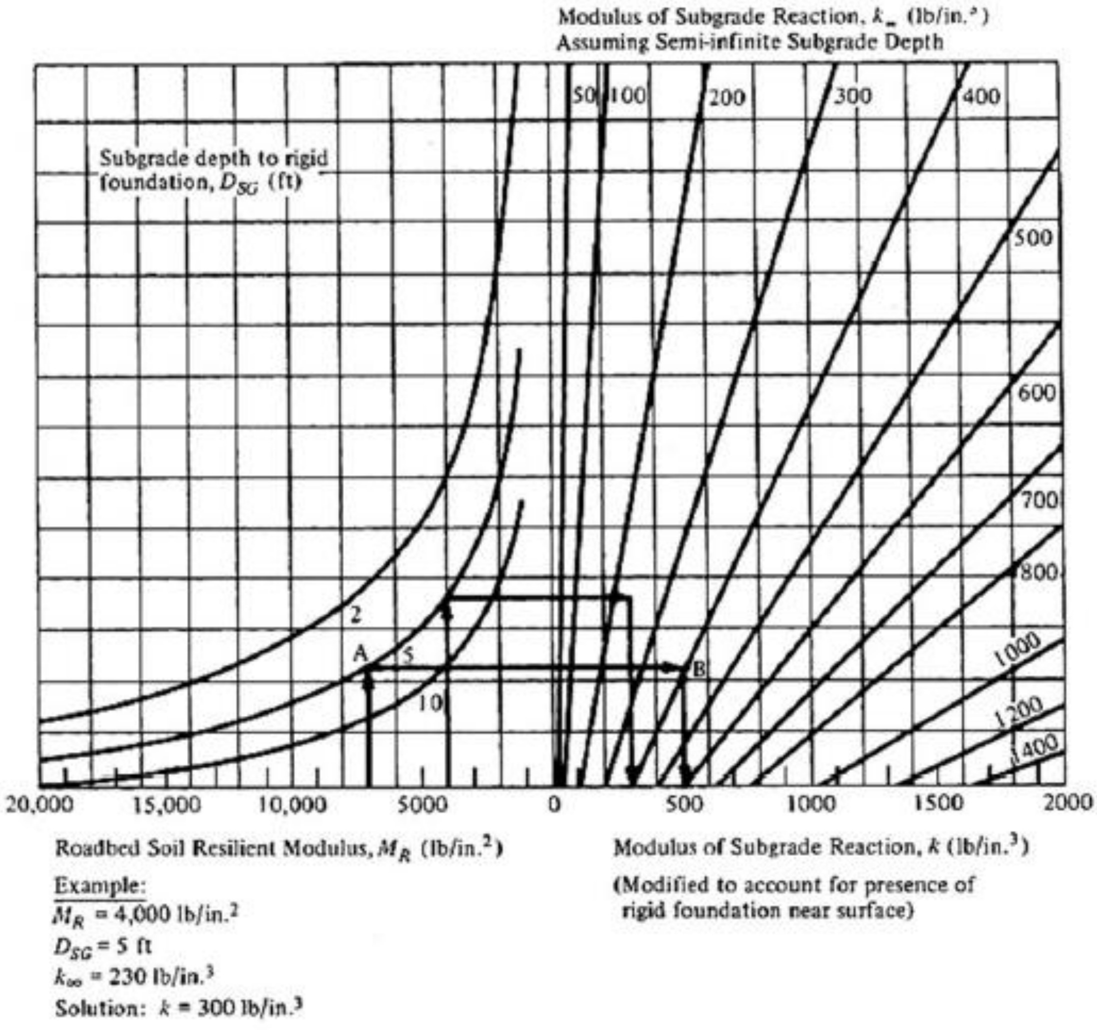

Modulus of Subgrade Reaction, k_∞ (lb/in.³)
Assuming Semi-infinite Subgrade Depth
50
100
200
300
400
500
600
700
800
1000
1200
1400
Subgrade depth to rigid foundation, D_{SG} (ft)
2
5
10
A
B
20,000
15,000
10,000
5000
0
500
1000
1500
2000
Roadbed Soil Resilient Modulus, M_R (lb/in.²)
Modulus of Subgrade Reaction, k (lb/in.³)
(Modified to account for presence of rigid foundation near surface)
Example:
M_R = 4,000 lb/in.²
D_{SG} = 5 ft
k_∞ = 230 lb/in.³
Solution: k = 300 lb/in.³

Figure 2.26

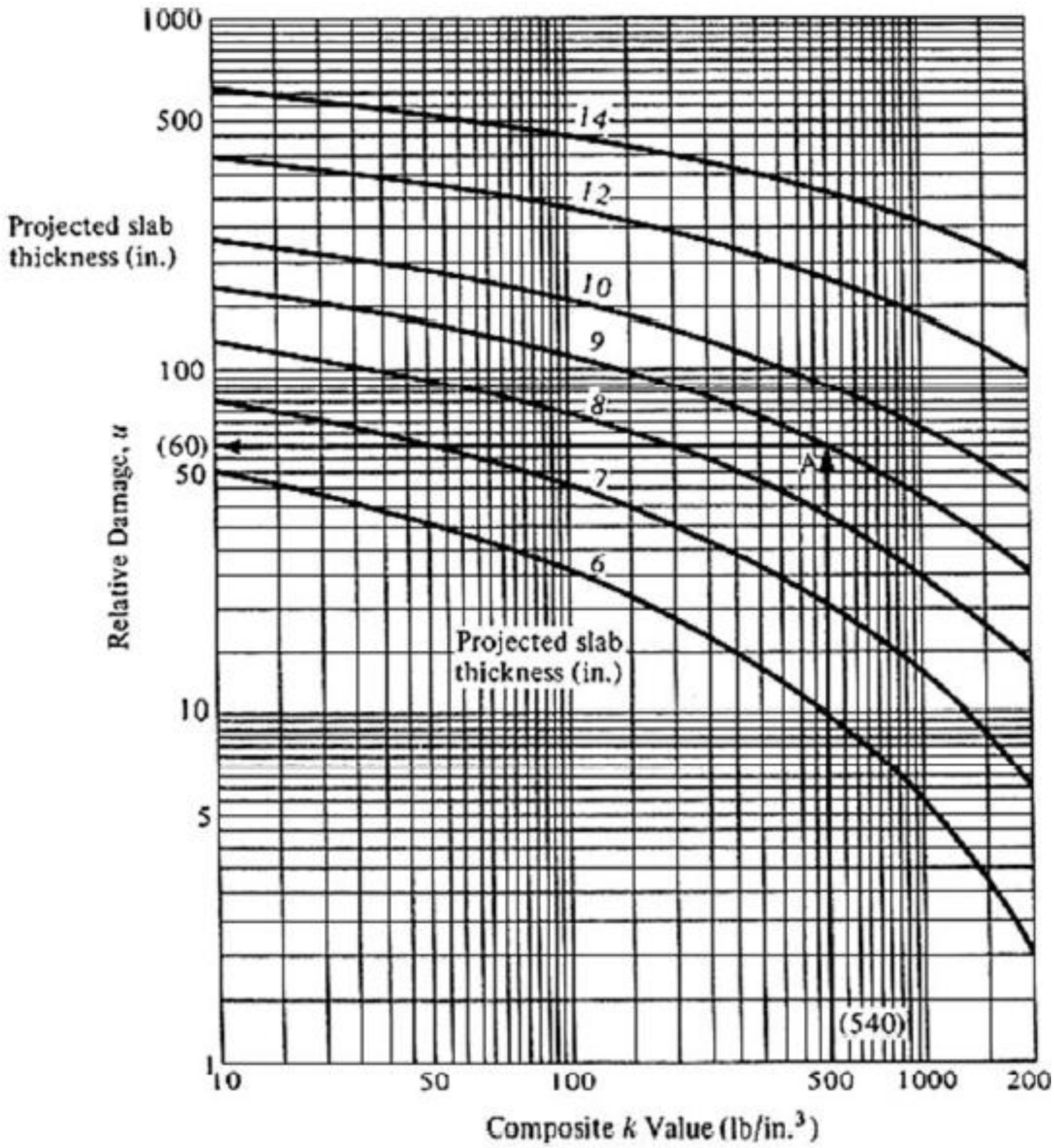


Figure 2.27: relation between composite K value and relative damage

Example: A 6 in. layer of cement-treated granular material is to be used as subbase for a rigid pavement. The monthly values for the roadbed soil resilient modulus and the subbase elastic (resilient) modulus are given in columns 2 and of Table 2.11. If the rock depth is located 5 ft. below the subgrade surface and the projected slab thickness is 9 in. Estimate the effective modulus of subgrade reaction using the AASHTO method

Table 2.11

Month	Roadbed Modulus M_r (Ib/in^2	Subbase Modulus E_{SB} (Ib/in^2	Composite k Value (Ib/in2) Figure 2.24	k Value (E_{SB}) on Rigid Foundation Figure (2.26)	Relative Damage u_r Figure (2.27)
Jan	20,000	50,000	**1100**	**1350**	**0.35**
Feb.	20,000	50,000	**1100**	**1350**	**0.35**
Mar.	2,500	15,000	**160**	**230**	**0.86**
Apr	4,000	15,000	**230**	**300**	**0.78**
May	4,000	15,000	**230**	**300**	**0.78**
Jun.	7,000	20,000	**400**	**500**	**0.6**
Jul.	7,000	20,000	**400**	**500**	**0.6**
Aug.	7,000	20,000	**400**	**500**	**0.6**
Sep	7,000	20,000	**400**	**500**	**0.6**
Oct.	7,000	20,000	**400**	**500**	**0.6**
Nov.	4,000	15,000	**230**	**300**	**0.78**
Dec.	20,000	15,000	**1100**	**1350**	**0.35**
Total					7.25

Type: Granular

Thickness of subbase (in) = 6

Loss of Support, L.S=1.0

Depth to rigid foundation (ft) =5

Projected Slab thickness (in) = 9

Average: $\bar{u}_r = \frac{\sum u_r}{n} = \frac{7.25}{12} = 0.6$

Therefore, Effective modulus of subgrade reaction k (Ib/in^2) =500

Corrected for loss of support: k (Ib/in^2) = 170

Concrete properties

Flexural strength (modulus of rupture) and elastic modulus at 28-day is used to represent the property of concrete.

Drainage

The concept of introducing the drainage into pavement design guide is similar to that discussed previously in flexible pavement design. However, the drainage coefficient (C_d) is determined from Table 2.12

Table 2.12: Recommended values for drainage coefficient Cd for rigid pavements

Quality of Drainage	*Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation* Less Than 1%	1–5%	5–25%	Greater Than 25%
Excellent	1.2–1.20	1.20–1.15	1.15–1.10	1.10
Good	1.20–1.15	1.15–1.10	1.10–1.00	1.00
Fair	1.15–1.10	1.10–1.00	1.00–0.90	0.90
Poor	1.10–1.00	1.00–0.90	0.90–0.80	0.80
Very poor	1.00–0.90	0.90–0.80	0.80–0.70	0.70

SOURCE: Adapted from *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C., 1993. Used with permission.

Reliability

This concept is as discussed in flexible pavement design

Structural pavement design

AASHTO pavement design guide suggest the following equation to determine the slab thickness

$$\log_{10} W_{18} = Z_R S_o + 7.35 \log_{10}(D+1) - 0.06 + \frac{\log_{10}[\Delta PSI/(4.5-1.5)]}{1+[(1.624\times 10^7)/(D+1)^{8.46}]} + (4.22 - 0.32P_t)\log_{10}\left\{\frac{S'_c C_d}{215.63J}\left(\frac{D^{.75}-1.132}{D^{.75}-[18.42/(E_c/k)^{.25}]}\right)\right\}$$

Where:

Z_R = standard normal variant corresponding to the selected level of reliability
S_o = overall standard deviation (see flexible pavement design)
W_{18} = predicted number of 18 kip ESAL applications that can be carried by the pavement structure after construction
D = thickness of concrete pavement to the nearest half-inch
ΔPSI = design serviceability loss = $p_i - p_t$
p_i = initial serviceability index
p_t = terminal serviceability index
E_c = elastic modulus of the concrete to be used in construction (lb/in^2)
S'_c = modulus of rupture of the concrete to be used in construction (lb/in^2)
J = load transfer coefficient = 3.2 (assumed)
C_d = drainage coefficient

The above equation can be solved to obtain the thickness (D) in inches by using either a computer program or the two charts in Figure 2.28 and Figure 2.29.

Example: Design a rigid pavement using AASHTO method using following Data:

Effective modulus of subgrade reaction, k = 72 Ib/in^3

Mean concrete modulus of rupture, S_c' = 650 Ib/in^2

Load transfer coefficient, J= 3.2

Drainage coefficient, C_d= 1.0

These values are used to determine a value on the match line as shown in Figure 2.28 (Segment 1), (Sold line ABCDEF)

Input parameters for segment 2 (Figure 2.29) on the chart are:

Match line value determined in segment 1 (74)

Design serviceability loss ΔPSI = 4.5-2.5=2

Reliability, *R%* =95% (Z_R= 1.645)

Overall standard deviation, $S_\circ$ = 0.29

Cumulative 18 kip ESAL = 5* 10^6

Based on above values, the required thickness slab is then obtained as shown in figure 2.29, as 10 in. (nearest half-inch)

Q: it is required to design a flexible pavement structure for a rural highway to carry a traffic load of 6*106, expressed in terms of ESAL. Experiemental results showed that the water takes about a month to drain from within the pavement structure. Weather forecasts indicated that the pavement structure may be saturated for 10% of the time. All other required information resulted from basic characterization is shown in figure below. Using ASSHTO 1993 pavement design guide to estimate thicknesses of pavement layers. Use P_i =4.5, p_t=2.5 and $S_\circ$=0.45