Hashemite University
Department of Civil Engineering

Highway Engineering and Design
CE 110401368

Lecture Notes

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2	Cross-section elements
3	Sight Distances
4	Horizontal Alignment
5	Vertical Alignment
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Geometric Design of Highways

Geometric design includes the design of geometric of cross sections elements, sight distance consideration, vertical alignments, horizontal alignments, intersection and various design details.
The American Association of State Highways and Transportation Officials (AASHTO) serves as a critical function in developing guidelines and standards used in highway geometric design.

Factors influencing highway design

Highway design is based on specified design standards and conflicts which depend on the following roadway system factors:

1- Functional classification
2- Design hourly volume & vehicle mix
3- Design speed
4- Design vehicle
5- Cross section element of highways
6- Topography
7- Presence of heavy vehicle on steep grades
8- Level of Service (LOS)
9- Environmental
10- Economic
11- Safety

Highway Functional Classification

Highways can be classified in many ways.

1- ***According to functions (based on speed and accessibility)***
   a. Freeways
   b. Arterial
   c. Collectors
   d. Local roads

Figure 1 illustrates the traditional hierarchy of these categories. The typical trip starts on a local street. The driver seeks closest collector, using it to access the nearest arterial. If the trip is long enough, a freeway or limited access is used.

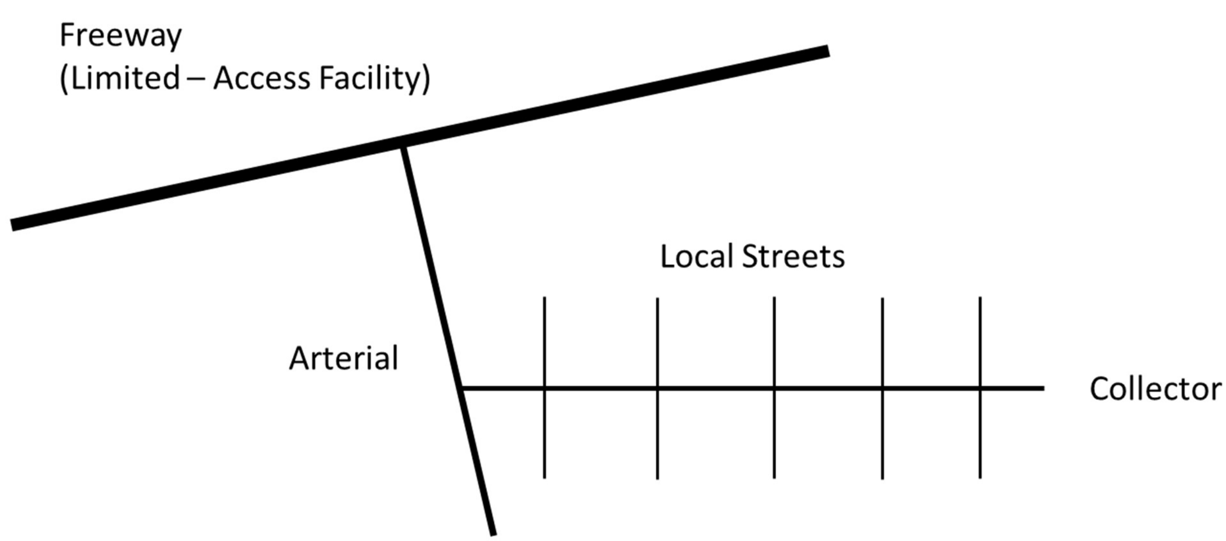


Figure 1: Hierarchy of Roadway Classifications

Urban roads are functionally classified into arterial, collector and local roads, as illustrated in figure 2.

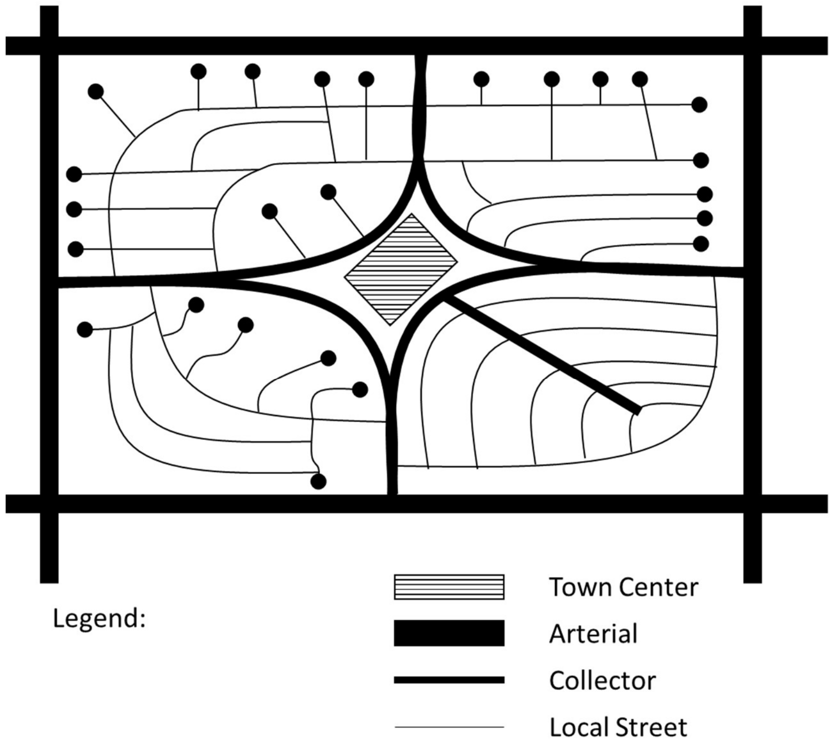


Figure 2: Schematic Illustration of Function Classes of Suburban Network

Freeways

They are access controlled divided highways, and are divided into three classes:

a. Very limited access
b. Partially limited
c. Free access

Most freeways are four lanes; two lanes in each direction or more.

Characteristics of Freeways

a. They are designed for high speeds – 120 km/hr
b. High traffic volume
c. Wide curves (radius is high)
d. Wide lanes
e. Barriers are located on both sides to prevent entering the road
f. Deceleration & acceleration lanes are provided
g. They are generally separated by interchanges at intersections (grade separated)
h. Exiting is always from the right
i. Expensive but capacity is high

Arterials

- They are generally divided highways with fully or partially controlled access.
- They are for through traffic & long distances.
- Parking, loading, and unloading are restricted.
- Pedestrians are allowed to cross only at intersections for pedestrian crossings.
- Design speed 80km/h

Collectors

- These roads are intended for collecting and distributing traffic to and from local streets and for providing access to arterial streets.
- There are few parking restrictions except during peak hours.
- Intermediate distances.
- Located in residential, business and industrial areas.
- Full access allowed
- Design speed 50km/h

Local

- They are intended for access to residences or businesses.
- Low traffic volume.
- Parking is allowed.
- Free movement for pedestrians.
- Design speed 30km/h

2- Classifications According to jurisdiction (responsibility)

a. Urban roads: responsibility of local authorities or municipalities
b. Rural roads: responsibility of Ministry of Public Works

Differences between rural and urban roads:

Urban	**Rural**
Land is expensive	Land is cheap
Access is more important than speed	Speed is more important than access
Design speed is low	Design speed is high
Pedestrian Crossing is important	Pedestrian Crossing is restricted
Heavy vehicles are not allowed	Heavy vehicles are important and allowed
Surface water Drainage is important	Surface water Drainage is less important
Loading and unloading allowed	No loading and unloading
Lanes are not wide	Lanes are wider
Cycling	No cycling
Level of Service (LOS) is low	LOS is high
More accidents	Fewer accidents

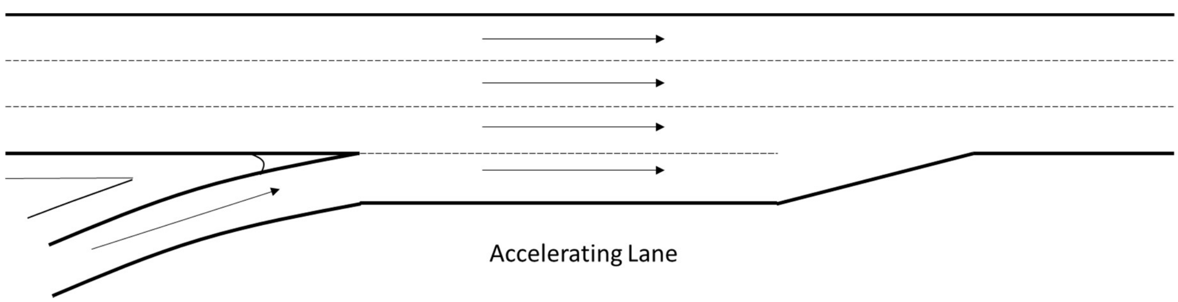
- Roads can be classified based on other criteria, such as traffic volume, land use and location.

Type of lanes:

a. Climbing Lane: Is an additional lane in the upgrade direction for heavy trucks travelling at a low speed, to enable vehicles to overtake the slow vehicles.

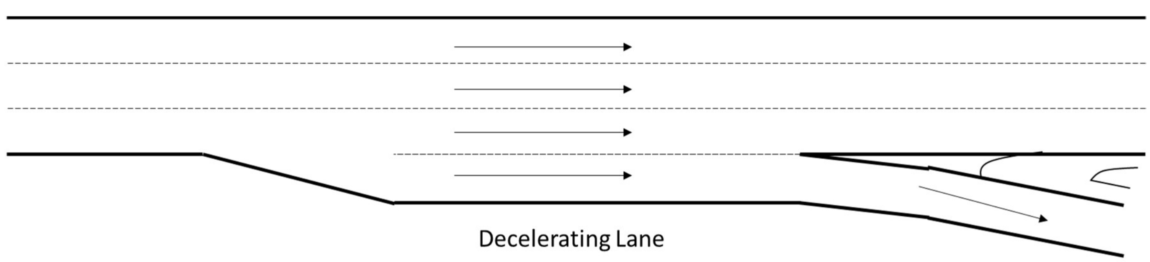
b. Escaping Lane: Is an emergency escape ramp provided on the downgrade of a Highway for use by trucks that have lost control and cannot slow down. A lane is provided that diverges when a vehicle enters the escape ramp.

c. Acceleration Lane: Is a speed change lane including tapered areas for the purpose of enabling a vehicle entering a road way to increase its speed to a rate at which it can more safely merge with through traffic.



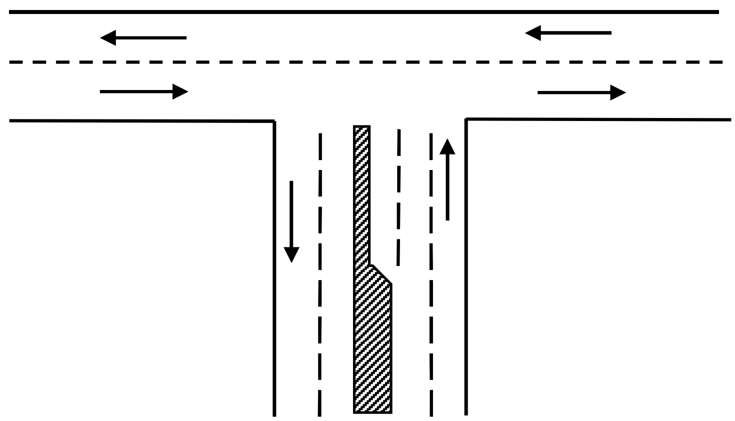
Accelerating Lane

d. Deceleration lane: Is a speed change lane including tapered areas for the purpose of enabling a vehicle exiting a road way to leave the travel lane and slow to a safe exit.



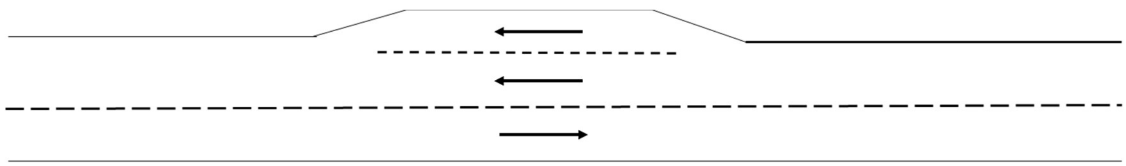
Decelerating Lane

e. Turning lanes (Storage lanes): Is a lane set aside for slowing down and making a turn, so as not to disturb traffic flow. By removing turning traffic from the through lanes drivers' safety is improved and delay is removed.

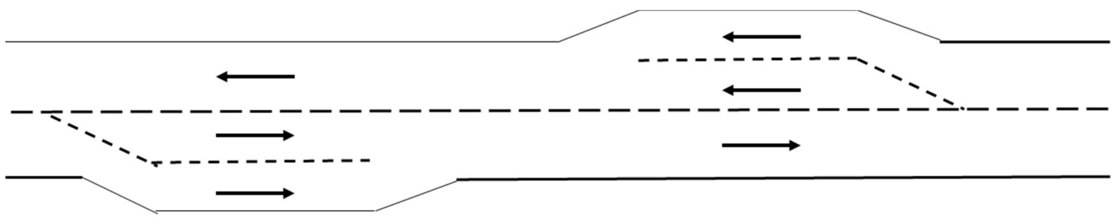


f. Overtaking Lane (Fast Lane): It is the lane furthest from the shoulder of a multi-lane roadway.

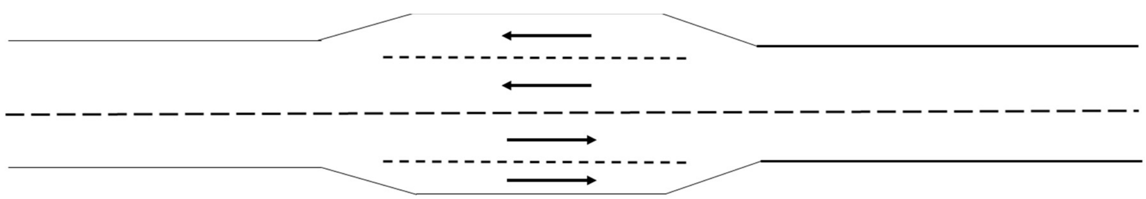
g. Passing Lane: Is an additional lane provided in one or both directions of travel on a two-lane highway to improve passing opportunities. They may internment or continuous lanes in level or rolling terrain and short four lane sections.



Isolated Lane Section



Intermittent Passing Lane



Short Four-Lane Section

h. Bus lane: Is a lane reserved for buses providing public transportation on a fixed route.
i. Cycle Lane
j. Main and Secondary lanes.

Design Speed

Design speed is defined by AASHTO as:

- the maximum safe speed that can be maintained over specified sections of highways when conditions are so favorable that the design features the highways govern.
  OR
- A selected speed to determine the various geometric features of the roadway.

Design speed depends on:

a. The functional classification of the highway
b. The topography of the area
c. The land use of the adjacent area
d. Traffic volume.

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.

The table below illustrates the minimum design speeds for various types of highways located in level, rolling and mountainous terrain.

Freeways	Design Speeds	
Terrain	**Rural**	**Urban**
Flat	70-80	70
Rolling	60-70	60-70
Mountainous	50-60	50-60

Arterial Highways		
Terrain	**Rural**	**Urban**
Flat	60-67	30-60
Rolling	40-60	30-50
Mountainous	30-50	30-50

Collector and Local Roads		
Terrain	**Rural**	**Urban**
Flat	30-50	30-40
Rolling	20-40	20-40
Mountainous	20-30	20-30

Source: Traffic Engineering Handbook (Fourth Edition), Institute of Transportation Engineers, Washington DC, 1992, p.156. Note: 1mile/hr

Design vehicle

Vehicles are classified by AASHTO into four main categories:

a. Passenger cars
b. Buses
c. Truck, single unit, semi-trailer
d. Recreational vehicles

The design vehicle is selected to represent all vehicles on the highway. Certain characteristics in the vehicle are important for the geometric design, the selected design vehicle is used to determine critical design features as described below:

The table below illustrates the relationships between vehicular and facility characteristics

Vehicular Characteristic	**Related Facility Characteristic**
LENGTH	Parking stall length Transit station platform length
WIDTH	Lane width Parking stall width Lateral clearance
HEIGHT	Vertical clearance Minimum vertical curve length
WHEELBASE (TURNING RADIUS)	Lateral clearance on curves Intersection edge radii
WEIGHT	Structural design of surface Structural design of bridges

The following guidelines apply when selecting a design vehicle:

1. Where a parking lot or a series of parking lots are the main traffic generators, the passenger car may be used.
2. For the design of intersection of local streets and park load a single unit truck may be used.
3. At intersections of state highways and city streets that serve buses with relatively few large truck, a city transit bus may be used.
4. At intersections of highways and low-volume county highways or township/local roads with less than 400 ADT, either an 84-passanger large school bus 12m long or a 65-passanger conventional bus 11m long may be used. The selection of either of these will depend on the expected of the facility.
5. At intersections of freeway ramp terminal and arterial crossroad, and at intersections of state highways and industrialized streets that carry high volumes of traffic, the minimum size of the design vehicle should be WB-20.

Topography

Highway design topography is generally classified into three groups:

a. Level terrain: < 10% slope
b. Rolling terrain: 10% - 25% slope
c. Mountainous terrain: >25% slope

Design hourly volume

The general unit of measuring traffic on highway is the annual average daily traffic abbreviated as (AADT). The traffic flow keeps fluctuating with time, from low value during off-peak hours to the highest volume during the peak hour.

In design, peak-hour volumes are studied from projection of the AADT. AADT are converted to a peak-hour volume in the peak direction of flow. This is referred to as the **Directional Design Hourly Volume (DDHV).**

DDHV = AADT * K * D

Where:
K = Represents the proportion of AADT occurring during the 30th peak hour of the year.
D = proportion of peak-hour travelling in the peak directions of flow.

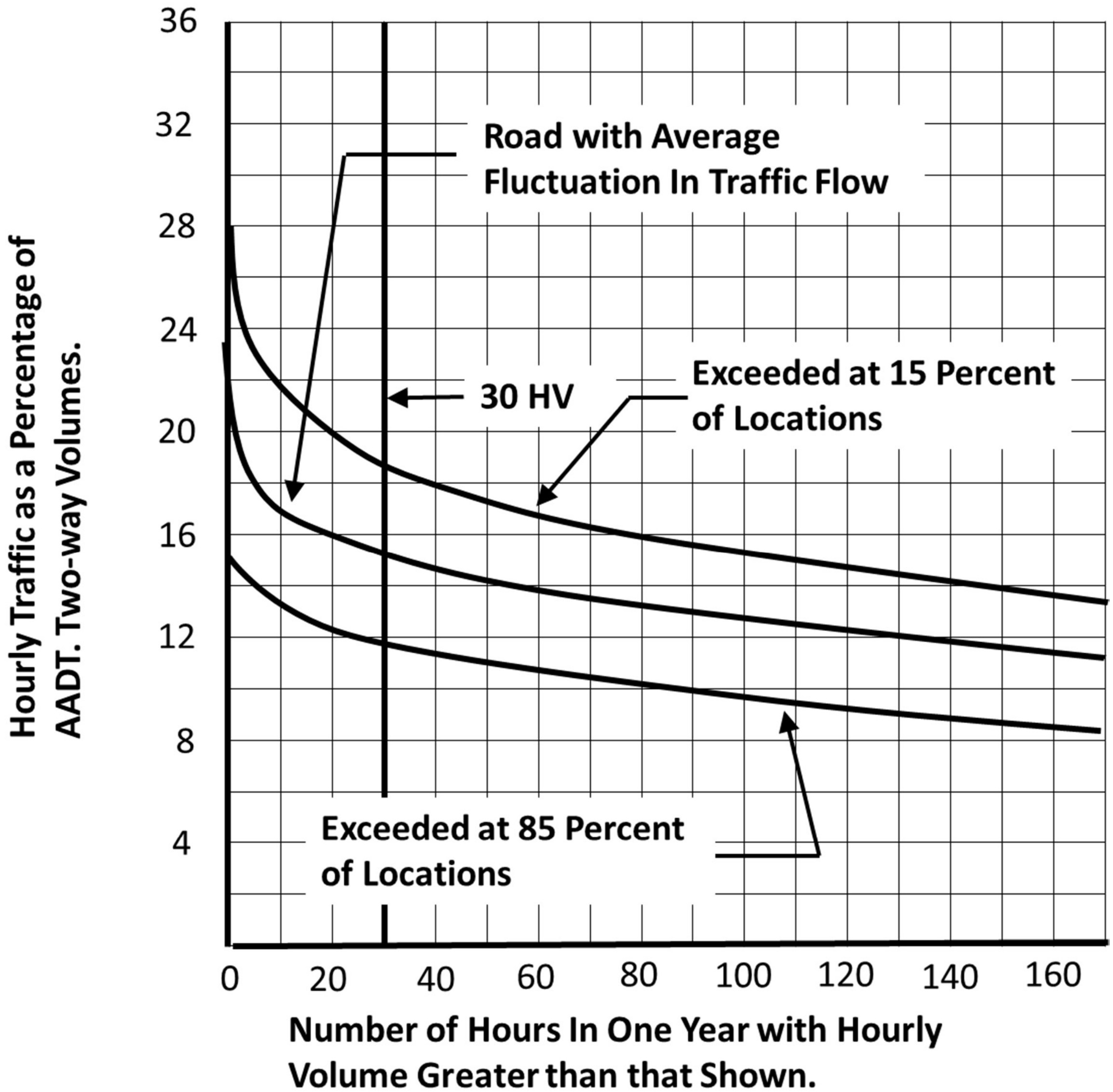
Example :

If the AADT = 10,000 vehicle for both directions of traffic flow. The percentage of the 30th hour is 20% of the AADT and the peak direction is 60%. Find the number of lanes if the lane capacity is 600 v/h.

Solution:

DDHV = AADT * K * D
DDHV = 60% * 20% * 10,000 = 1200 v/h
Therefore, the number of lanes = 1200/600 = 2 lanes required

The road is 2 lanes in each direction so it’s a 4-lane highway.



Hourly Traffic as a Percentage of AADT. Two-way Volumes.
36
32
28
24
20
16
12
8
4
Road with Average Fluctuation In Traffic Flow
30 HV
Exceeded at 15 Percent of Locations
Exceeded at 85 Percent of Locations
0
20
40
60
80
100
120
140
160
Number of Hours In One Year with Hourly Volume Greater than that Shown.

Cross-section elements of highway

The cross section of highway consists of the following:

1. Travel lanes
2. Medians
3. Shoulders
4. Kerb stones and gutters
5. Guardrails (barriers)
6. Drainage channels
7. Side slopes
8. Right of Way
9. Formation width
10. Sidewalk
11. Cross slopes (Pavement crown)

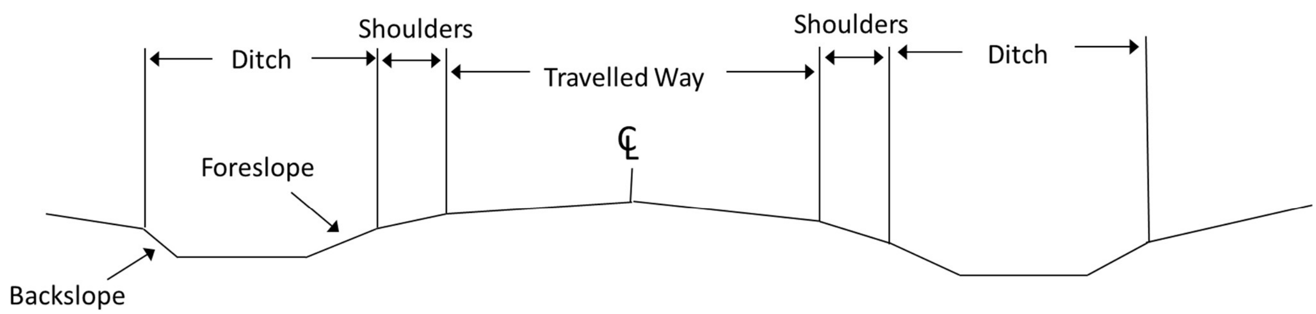


Figure 3: Undivided Two-lane highway cross section, with ditches

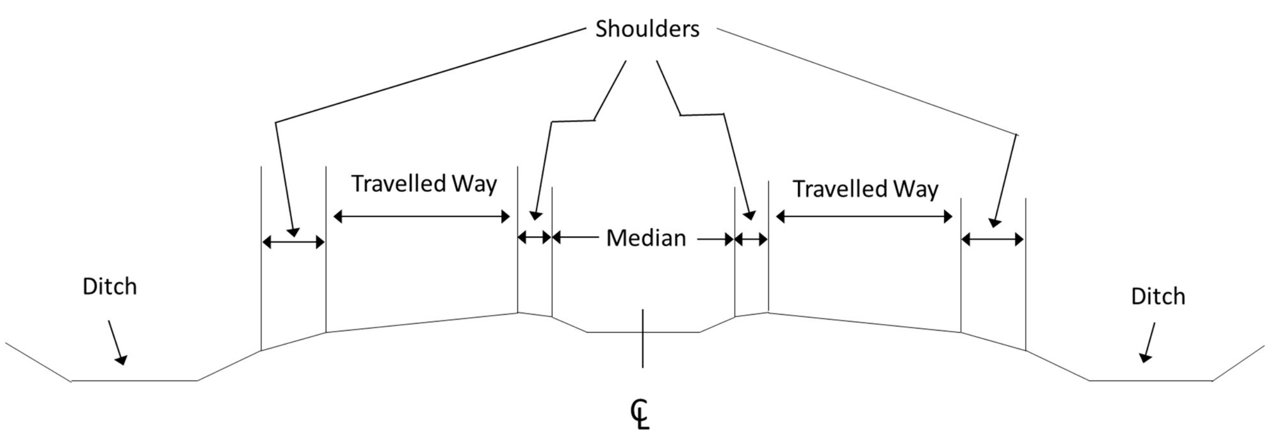


Figure 4: Divided highway cross section, depressed median with ditches.

1. Travel lanes

a. The standard width in 3.6m (12ft) and the minimum is 2.7m (9ft)
b. Lanes wider than 3.6m are provided at curves to account for the rear wheel of large vehicles
c. Lanes that are 2.7m (9ft) wide are used in urban areas. If traffic volume is low and low speed roads
d. Lanes less than 3.3m increase accidents for trucks and heavy vehicles

The width and number of lanes depends on:

a. Volume of traffic
b. The design speed

Category of Highways

1. Two-lane highway (width 7.2m) with shoulders on each side
2. Three-lane highway
    - Three-Lane highway may be used in the following cases
        a) 2-lane in one direction and one-lane in the other (permanent)
        b) The third lane may be used alternately
        c) For climbing lanes
        d) For left-turners only (urban areas) (center lane)

3 four-lane highway, it should be divided.

2 Medians

It is the section of a divided highway with four or more lanes that separates the lanes in opposing directions.

The function of a median includes:

- Providing a recovery area for out-of-control vehicles
- Separating opposing traffic
- Providing storage areas for left-turning and u-turning vehicles

- Providing refuge for pedestrians
- Reducing the effect of headlight glare

Medians can be:

  a. Raised – Raised medians are used in urban roads
  b. Flush – Flush medians are used in urban roads and in freeways with median barriers
  c. Depressed – Depressed medians are used in freeways, and are more effective in draining surface water.

- Minimum width of 10ft (3m) is recommended for 4-lane urban freeways
- .A minimum of 6.6m (22ft) is recommended for six or more lanes of freeway.
- Median width for urban collector streets vary from 0.6 to 12 m (2 – 40 ft) depending on the median treatment.
- Median widths vary from 1.2m to 24m or more
- A side slope of 6:1 is common

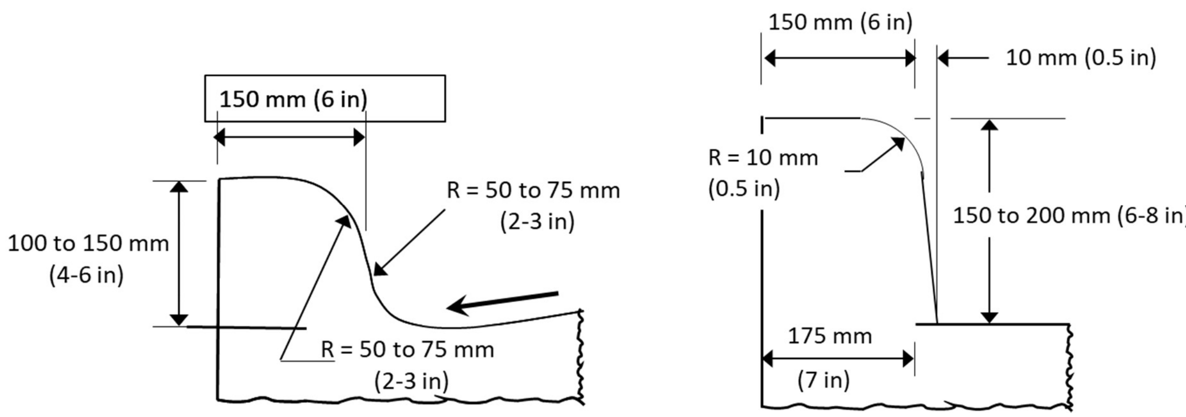
3 Shoulders

Are the strips provided on both sides of the carriageway?

- They are provided for safe operation of traffic.
- They increase sight distance on Horizontal curves.
- They provided a refuge for stalled vehicles or emergencies.
- They provided structural support for the Pavement.
- Improve capacity. A good shoulder will increase effective pavement width by 0.6m
- Width should be >3m.
- In mountainous areas >1.2m
- Slope >3% to carry water away from the highway.
- Surface should be rough compared to the main road. As a safety measure to warn drivers that they are leaving the traffic lane.

4 Kerb Stone and Gutters

- Kerbs are made of concrete
- They are mainly used in urban areas to:
  a) delineate pavement edges
  b) Control Drainage
  c) Delineate of pedestrian side walk.
- Kerbs should not be too high, so as to allow for the opening of car doors, but not so low as encourage cars to drive over pavements (15-20cm)



The figure above shows Vertical and Sloping Kerbs the type that are provided for urban streets.

Gutters

Are usually located on the pavement side of a Kerb to provide the principal drainage facility for the highway.

6 Guard Rails (Barriers)

They are used to minimize the severity of accidents that might occur. Guard rails are longitudinal barriers, and are provided in the following cases:

1. When fill is > 2.4m
2. Sudden change in alignment (sharp horizontal curves)
3. In locations where there are deep road side ditches

Guard rails should be flexible to reduce damage if there are any accidents, and to absorb the energy due to impact.

Types of Guard Rails:

1. W-Beam (rail and posts) semi flexible.

2. Cables (flexible)

3. Concrete Barriers. (rigid)

6 Drainage Ditches:

- Longitudinal channels (ditches) are constructed along the sides of a highway to collect the surface water from the pavement surface.
- Drainage ditches are in cut sections.
- Flat bottomed shaped are preferred over v-shaped.

7 Side Slopes:

Side slopes are provided on embankment and fills to provide stability for earthwork. They also serve as a safety feature providing a recovery area for out of control vehicles.

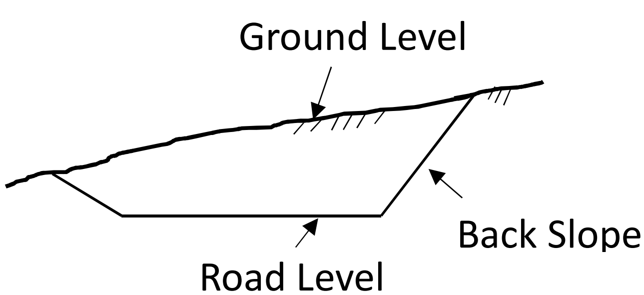
Steep slopes will result in reducing earthwork, but steep slopes are also more liable to erosion than flatter slopes

Side slopes usually 4:1 when fill is about 3m (H:V)
And 6:1 when fill is about 1.8m

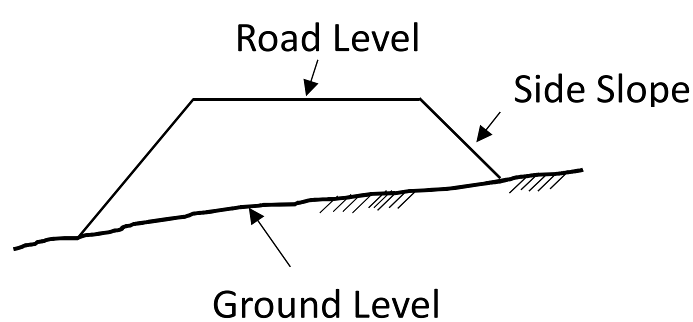
Back slopes are normally 2H:1V

Rock cuts are usually stable at steep slopes depending on the homogeneity of the rock.

In some cases, to avoid erosion, side slopes are built with rip-rap



Cut Section



Fill Section

8 Right of Way:

It is the total land area acquired along the road alignment by the highway organization. The width should be sufficient to accommodate all the elements of the highway cross-section

The Right of Way depends on the importance of the road. Sufficient Row should be acquired in order to:

1. Avoid the expense of purchasing developed properties
2. Provide drainage system
3. For side slope of embankment or cutting
4. Visibility consideration on curves
5. Width of land required for future development

Recommended values for Row

- For 2-lane urban collector = 12-18m
- 2-lane arterial = 25m
- 4-lane arterial (undivided) = 19-33m
- 4-Lane arterial (divided) = 36-90m

9 Formation Width

- It is the sum of widths of a highway shoulder and median if provided.
- In case of embankments, it is measured as the top width
- In case of cutting, it is measured as the bottom width of the cutting from which side drains are excluded.

10 Side Walks

- They are provided on roads in urban areas on both sides, but are uncommon in rural area and when pedestrian traffic is high in urban or rural areas.
- They are also provided on arterial roads when no shoulders are provided

- The minimum width is 4ft (1.2m) in residential areas, and range from 4-8ft in commercial areas.

11 Cross Slopes (Pavement crown).

- Pavements on straight sections of two-lane and multi-lane highways without medians are sloped from the middle downwards to both sides of the highway, resulting in a traverse 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.
- Cross slopes on divided highways are provided by either crowning the pavement in each direction, as shown in figure 15.9(a), or by sloping the entire pavement in one direction, as shown in figure 15.9(b).

- The advantage of sloping the pavement in each direction is that surface water is quickly drained away from the travelled roadway during heavy storms.
- The disadvantage is that additional drainage facilities, such as inlets and underground drains, are required. This method is mainly used at areas with heavy rain and snow.
- In determining the rate of cross slope for design, two conflicting factors should be considered. Although a steep cross slope is required for drainage purposes, it may be undesirable since vehicles will tend to drift to the edge of the pavement, particularly under icy conditions.
- Recommended rates of cross slope are 1.5 to 2 percent for high-type pavements and 2 to 6 percent for low-type pavements.

Sight Distance

Sight distance is the length of the roadway a driver can see ahead at any particular time. The sight distance available at each point of the highway must be such that, when a driver is travelling at the highway's design speed, adequate time is given after an object is observed in the vehicle path to make the necessary maneuvers without colliding with the object. There are two types of sight distance:

a Stopping Sight Distance (SSD)
b Passing Sight Distance (PSD)

Stopping Sight Distance
The Stopping Sight Distance (SSD) for design purposes, is usually taken as the minimum sight distance required for a driver to stop a vehicle after seeing an object in the vehicles path without hitting that object.

The SSD depends on:
a. Reaction time and
b. The breaking distance

SSD = dr + db
dr = the distance traveled during reaction time
db = the breaking distance

$$SSD = 0.278\, v * t_r + \frac{v^2}{254\,(f \pm G)}$$

$d_r = v * t_r \quad metre$ $where\ v\ in\ m/sec.$ $t_r = 2.5$	**OR**	d_r = 0.278 $v *\ t$ metre $where\ v\ in\ km/hr.$ $t_r = 2.5\ sec$

$$d_b = \frac{v^2}{2_g\ (f \pm G)}\ meters$$

$v\ in\ m/s$

$$d_b = \frac{v^2}{254\ (f \pm G)}\ metres$$

$v\ in\ km/h$

$G = Slope$

$f = friction\ coefficient$

The table below shows the Friction Coefficient recommended by AASHTO

Design Speed km/h	30	40	50	60	70	80	90	100	110	120
Friction Coefficient	0.4	0.38	0.35	0.33	0.31	0.30	0.30	0.29	0.28	0.28

Example 1:

Determine the minimum stopping distance on a – 3.5% grade for a design speed of 110km/h.

$$SSD = d_r + d_b$$

Reaction distance:

$$d_r = v * t_r = (110 \text{ km/h}) \left(\frac{1{,}000 \text{ m/km}}{3{,}600 \text{ s/h}}\right) (2.5 \text{ s}) = 76.4\text{m}$$

Breaking distance:

$f = 0.28$ (from the table)
$G = 0.035$ (given)

$$d_b = \frac{v^2}{2g\,(f \pm G)} = \frac{\left[(110 km/h)\left(\frac{1{,}000 m/km}{3{,}600 \ s/h}\right)\right]^2}{2(9.8 \ m/s^2)\ (0.28-0.035)} = 194.4m$$

Total Stopping Sight distance:

$$SSD = d_r + d_b = 76.4 + 194.4 = 270.8m$$

Exit Ramp Stopping Distance

Example 2:
A motorist travelling at 105 km/h on a freeway intends to leave the freeway using an exit ramp with a maximum speed of 56 km/h. At what point on the freeway should the motorist apply the brakes in order to reduce he speed to the maximum allowable on the ramp just before entering the ramp, if this section of the freeway has a downgrade of 3%.

Solution

$$D_b = \frac{u_1^2 - u_2^2}{254\left(\frac{a}{g} - 0.03\right)}$$

$$a/g \ = 3.41/9.81 \ = 0.35$$

$$D_b = \frac{105^2 - 56^2}{254\ (0.35 - 0.03)} = 97.06m$$

The brakes should be applied at least 97.06m from the ramp.

Where a = deceleration of the vehicle

Passing Sight Distance (PSD)

Passing Sight Distance is the minimum sight distance required on a two-lane two-way highway, that will permit a driver to complete a passing maneuver without colliding with an opposing vehicle and without cutting off the passed vehicle.

In order to determine the minimum passing sight distance, certain assumptions have to be made regarding the movement of the passing vehicle during a passing maneuver:

1. The vehicle being passed (impeder) is travelling at a uniform speed.
2. The speed of the passing vehicle is reduced and is behind the impeder as the passing section is entered.
3. On arrival at a passing section, sometime elapses during which the driver decides whether to undertake the passing maneuver.
4. If the decision is made to pass, the passing vehicle is accelerated during the passing maneuver, and the average passing speed is about 16km/h more than the speed of the impeder.
5. A suitable clearance exists between the passing vehicle and any opposing vehicle when the passing vehicle re-enters the right lane.

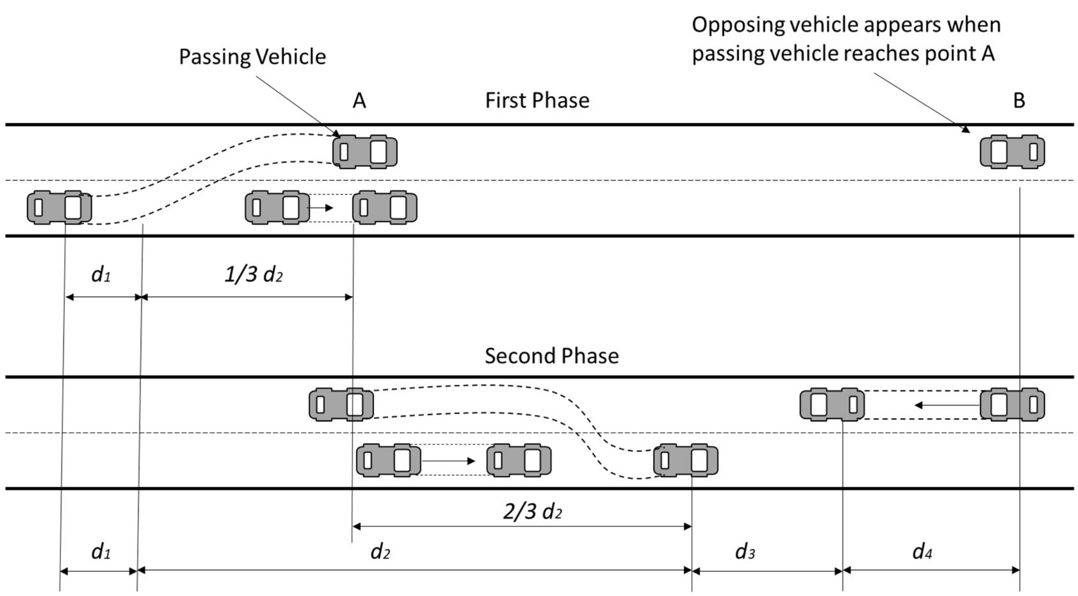
These assumptions have been used by AASHTO to develop a minimum passing sight distance requirement for two-lane, two-way highways.

The minimum passing sight distance is the total of four components as shown in the figure below.
The total passing sight distance (PSD) is equal to: -

$$PSD = d_1 + d_2 + d_3 + d_4$$

Where:

d_1 = distance traversed during perception-reaction time and during initial acceleration to the point where the passing vehicle just enters the left lane.
d_2 = distance travelled during the time the passing vehicle is travelling in the left lane.
d_3 = distance between the passing vehicle and the opposing vehicle at the end of the passing maneuver.
d_4 = 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)



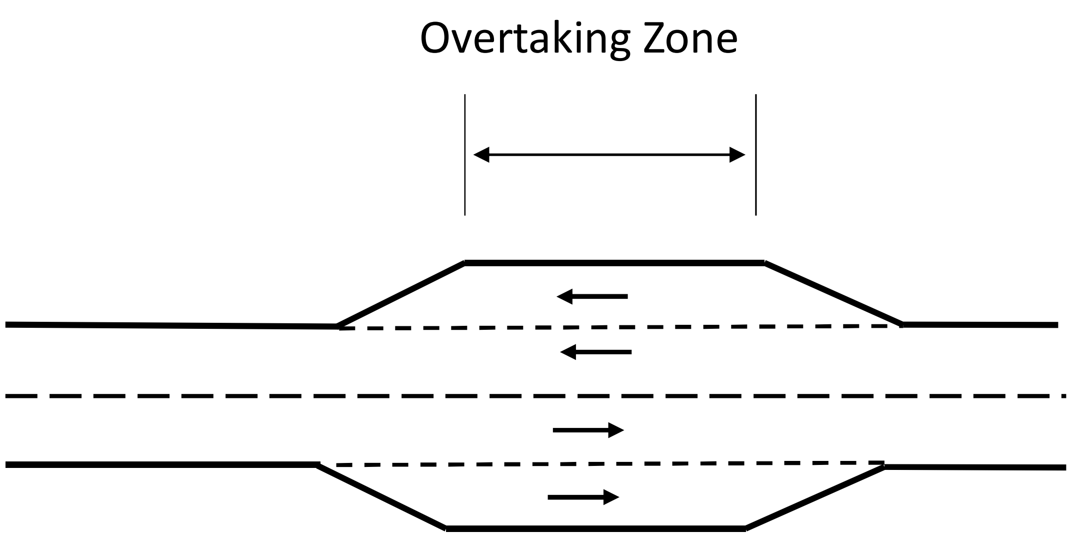
The passing sight distance recommended by AASHTO are shown in the table below

Design Speed km/h	30	40	50	60	70	80	90	100	110	120
PSD m	217	285	345	407	482	541	605	670	728	792

Overtaking Zones

In some places where it is difficult to provide passing sight distance throughout the length of the highway overtaking zones are provided for safe passing.

The average length of the overtaking zone is four times the PSD.



Highway Surveys and Locations

Before a highway alignment is finalized in a highway project, the engineering surveys are to be carried out. The highway location process involves four phases:

1. Office study of existing information. (Map study)
2. Reconnaissance survey.
3. Preliminary survey.
4. Final location and detailed surveys.

Office Study of existing information.

Data of the highway location are collected from the engineering reports, maps and the aerial photographs of the area.

The data obtained should be on the following characteristics of the area.

1. Engineering, including topography, geology and climate.
2. Social and demographic, including land use and zoning patterns.
3. Environmental, including types of wildlife, location of recreational, historical and archeological sites.
4. Economic, including unit costs for construction.

From this data, alternative routes can be suggested in the office. The probable alignment can be located on the map.

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

Preliminary analysis will identify unsuitable sites for the highway such as : Historical site, Ruins, Water sources and Buildings.

Reconnaissance Survey

The object of this phase of the study is to identify several feasible routes using aerial photographs, taking into account the following:

1. Topography and soil conditions.
2. Serviceability of route to industrial and population areas.
3. Crossing of other transportation facilities, such as rivers railroads and highways.
4. Directness of route.

Control points between the two endpoints are determined for each feasible route. For example, a unique bridge site with no alternative may be taken as a primary control point.

The most feasible routes identified are then plotted on photographic base map.

Preliminary Survey:

Objectives of preliminary surveys are:

1. To survey the various alternative alignments proposed after the reconnaissance and to collect all necessary physical information and detail of topography, drainage & soil.
2. To estimate quantity of earthwork materials and other construction aspects and to work out the cost of the alternative proposals.
3. To study the environmental impact of each alternative route

Prepare a report on the advantages and disadvantages of different alternative routes. As a result, a few alternative alignments may be chosen.

Final Location Survey

- The final location survey is a detailed layout of the selected route (Transferring the alignment on to the ground). The horizontal and vertical alignments are determined, and the positions of the structures and drainage channels are located

- Set the points of the intersections (**PI**) of the straight portions of the highway and fit a suitable horizontal curve between these.

- Center line stacks are driven at suitable intervals, say 50m intervals in plane and rolling terrain and 20m in hilly terrain.

Right-of-way Acquisition.

One factor that significantly affects the location of highways in urban areas is the cost of acquiring right of way. This cost is largely dependent on the predominate land use in the right of way of the proposed highway. Costs tend to be much higher in commercial areas, and landowners in these areas are often unwilling to give up their property for highway construction.

Principles of bridge location

The basic principle for locating highway location determines bridge; location, not the reverse. When the bridge is located first in most cases the resulting highway alignment is not the best. The general procedure for most highways therefore is to first determine the best highway location and then determine the bridge site. Only in cases where bridges need to be skewed or foundation problems exist, the location of the bridge can be a factor in highway location due to higher costs associated with the above-mentioned bridge conditions.

A detailed report should be prepared for the bridge site selected to determine whether there are any factors that make the site unacceptable. This report should include, accurate data on soil stratification, the engineering properties of each soil stratum at the location, the crushing strength of bedrock, and water levels in the channel or waterway

Maximum highway grades……..

- The selection of maximum grades for highway depends on the design speed and the design vehicle.
- Grades of 4 to 5 percent have little or no effect on passenger cars as the grade increases above 5%. however, speeds of passenger cars decrease on upgrades and increase on downgrade.
- Steep grades affect not only the performance of heavy vehicles, but also the performance of passenger cars. In order to limit the effect of grades on vehicular operation, the maximum grade on any highway should be selected properly.
- Grade has a greater impact on trucks than on passenger cars. Extensive studies have been conducted, and results have shown that truck speed may increase by up to 5% on down grades, and decrease by 7% on upgrades, depending on the percent and length of the grade.
- Maximum grades have been established based on the operating characteristics of the design vehicle on the highway. These vary from 5 percent for a design speed of 112km/h to between 7 and 12 percent for a design speed of 48km, depending on the type of highway. Table 15.4 gives recommended values of maximum grades.

Minimum Highway Grades

- Minimum grades depend on the drainage conditions of the highway. Zero percent grades may be used on uncurbed pavement with adequate cross slopes to laterally train the surface water.
- When pavements are curbed**,** however, a longitudinal grade should be provided to facilitate the longitudinal glow of the surface water.
- An appropriate minimum grade is typically 0.5%, but grades of 0.3% may be used where there is a high-type pavement accurately sloped and supported on firm grade.

TABLE 15.4

Type of Terrain	Rural Collectors Design Speed (mi/h)								
	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

Type of Terrain	Urban Collectors Design Speed (mi/h)								
	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

Type of Terrain	Rural Arterials Design Speed (mi/h)								
	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 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	-	-

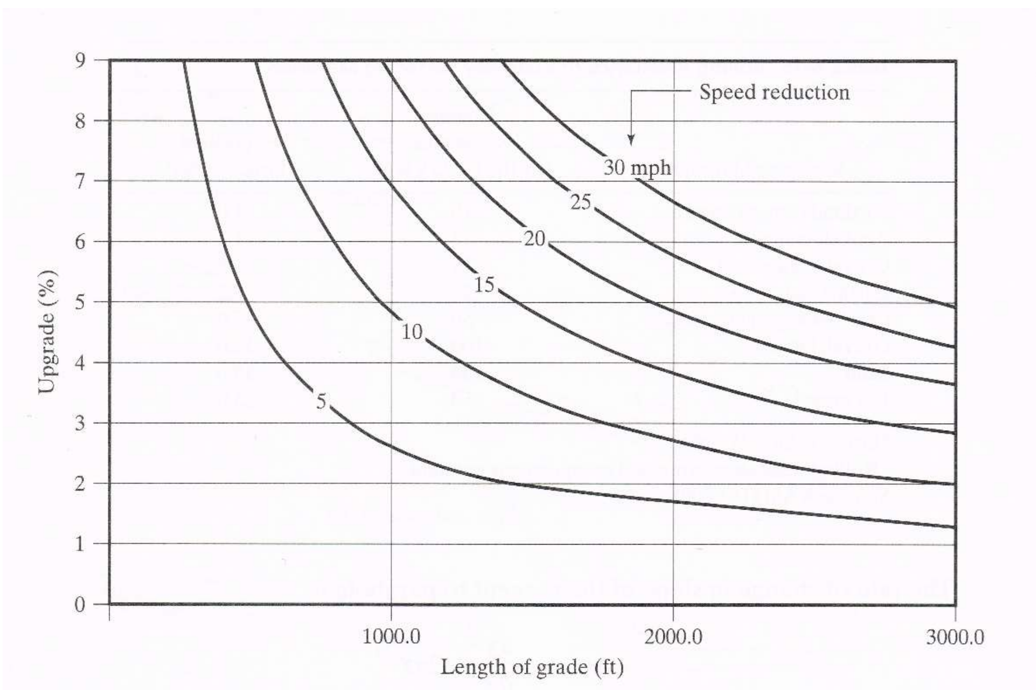
	Urban Arterials Design Speed (mi/h)						
Type of Terrain	30	35	40	45	50	65	80
				Grades (%)			
Level	8	7	7	6	6	5	5
Rolling	9	8	8	7	7	6	6
Mountainous	11	10	10	9	9	8	8

Critical Length.

For most design purposes, grades should not be longer than the “critical length”. The critical length is generally defined as the length at which the speed of trucks is 15 mi/h less than their speed upon entering the grade. When trucks enter an upgrade from slower speed, a reduction of 10 mi/h may be used to define the critical length of grade.

Example:
A rural freeway with rolling terrain has a design speed of 60 mi/h. What is the longest and steepest that should be included in the facility.

Solution:
From the tables above for a freeway facility with a design speed of 60 mi/h in rolling terrain the maximum allowable grade is 4 percent. Entering the figure above with 4 percent on the vertical axis, moving to the “15 mi/h” curve the critical length of grade is seen to approximately 1900ft.



Critical Length of Grade for a Typical Truck

Geometric Design

The Geometric Design of a highway is composed of Vertical and Horizontal alignment, and the cross-section element.

Below are some design standards highways which must be considered when designing highways.

- Minimum radius of horizontal curves: -

For given design speed, minimum curve radius is limited by the maximum allowable side friction which is usually based on a comfort standard, maximum superelevation (banking) for the curve and the necessity to maintain stopping sight distance.

- Maximum rate of superelevation:

Maximum superelevation rate is limited by side friction, this is to prevent slow moving vehicles from sliding to the inside of the curve under slippery conditions.

- Maximum Grades:

Most upgrades are limited to vehicle power/weight ratio and most down grades are limited by stopping sight distance.

- Minimum Grades:

These are limited by the need to provide drainage. A zero percent grade maybe used on uncurbed pavement with adequate cross slopes and lateral drainage of surface water.

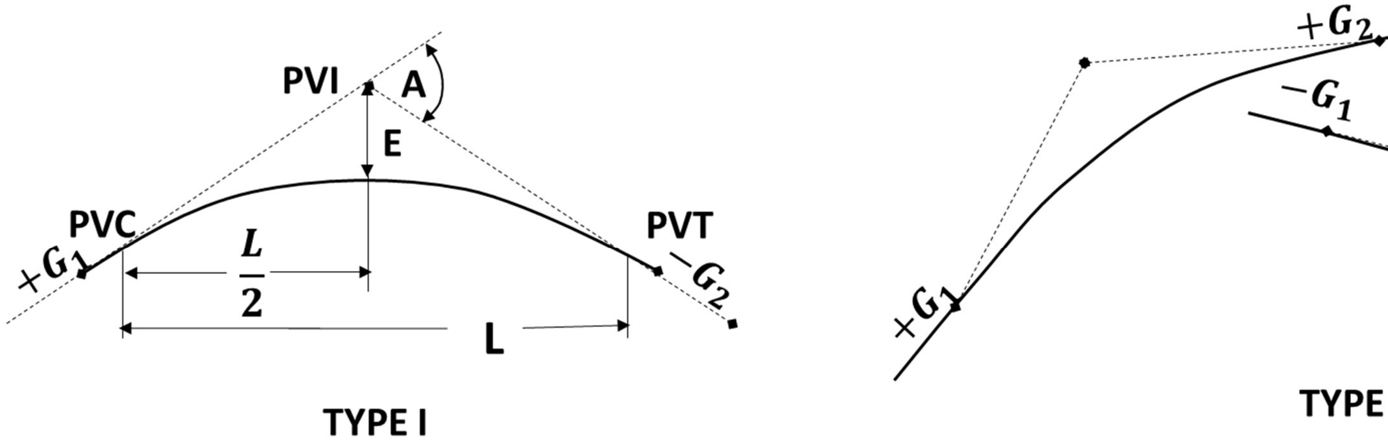
- Minimum Length of vertical curve:

This is limited by stopping sight distance, passing sight distance and appearance.

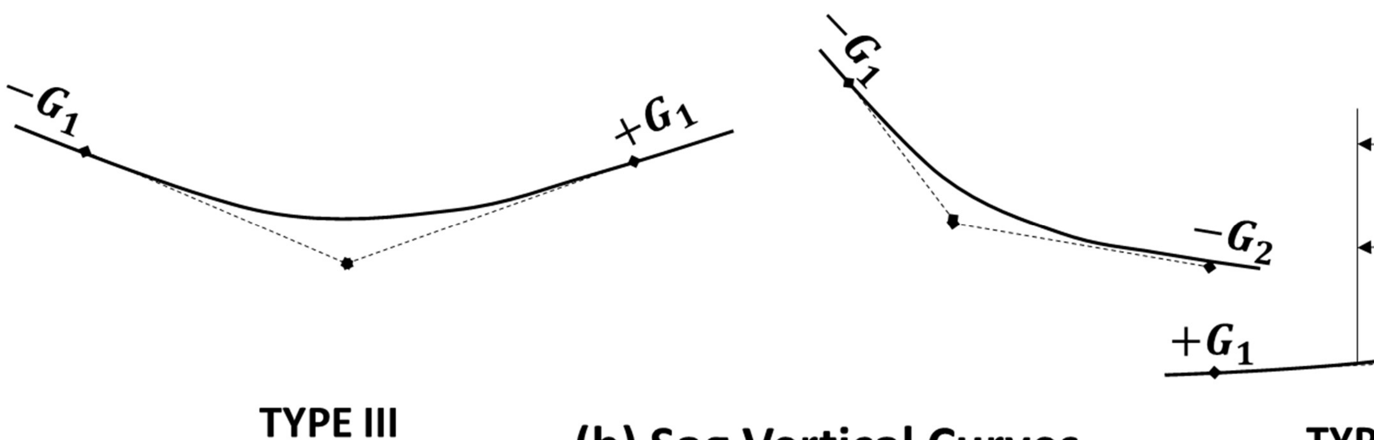
Design of the Alignment

Vertical Alignment

The vertical alignment of a highway consists of straight sections known as Grades, (or Tangents) connected by Vertical Curves. The design of the vertical alignment therefore involves the selection of Suitable Grades for the tangent sections and the appropriate length of vertical curves. Vertical curves are used to provide a gradual change from one tangent grade to another so that vehicles may run smoothly as they transverse the highway. These curves are usually parabolic in shape and classified as Crest or Sag Vertical Curves.



(a) Crest Vertical Curves



(b) Sag Vertical Curves

Types of Vertical Curves

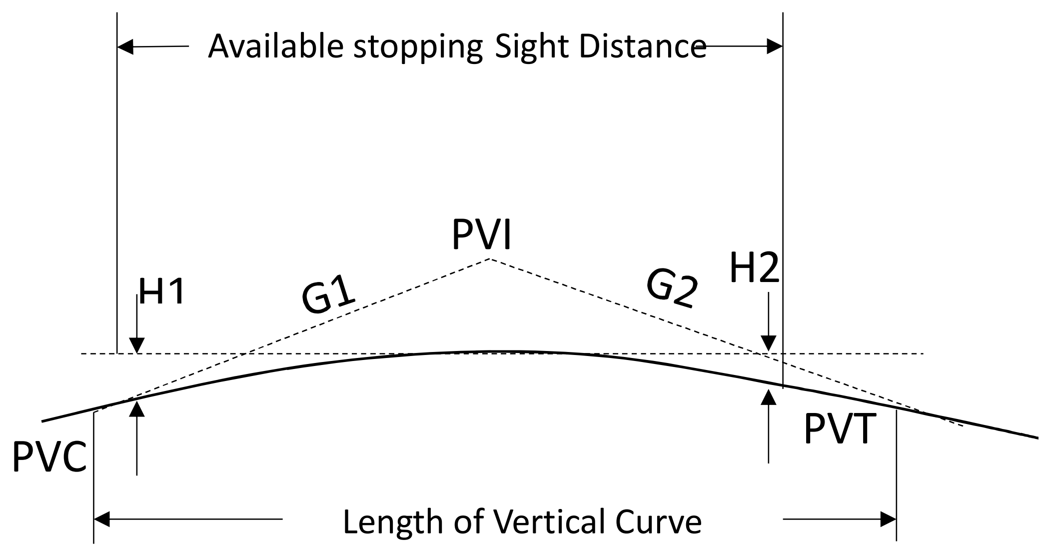
G_1, G_2 = Grades of Tangents (%)
A = Algebraic Difference.
L = Length of Vertical Curve.
PVC = Point of Vertical Curve.
PVI = Point of Vertical Intersection.
PVT = Point of Vertical Tangent.

Length of crest vertical curves

- Use of a minimum stopping sight distance (SSD) and passing sight distance (PSD) is the only criterion used for design of a crest vertical curve.

Stopping Sight Distance (SSD) and Crest Vertical Curve Design.

In providing sufficient SSD on a vertical curve, the length of curve (L) is the critical concern. Longer lengths of curve provide more SSD, all else being equal but are more costly to construct. Shorter curve lengths are relatively inexpensive to construct but may not provide adequate SSD. What is needed then is an expression for minimum curve length given a required SSD.:



Stopping Sight Distance (SSD)

- The stopping sight distance is comprised of the distance to perceive and react to a condition plus the distance to stop;’

$$SSD = 0.278\, V * t\, r + \frac{V^2}{254\,(\frac{a}{9.81} \pm G)} \quad (Metric.)$$

$$SSD = 0.278\, V * t_r + \frac{V^2}{254\,(\, f \pm G)} \quad (Metric.)$$

Where:

SSD = Required stopping sight distance, m.

V = Speed, km/h

t = Perception-reaction time, sec, typically 2.5 sec. for design.

f = Coefficient of friction, typically for a poor, wet pavement.

G = Grade decimal.

a = Deceleration rate, m/sec^2 (3.4 m/sec^2)

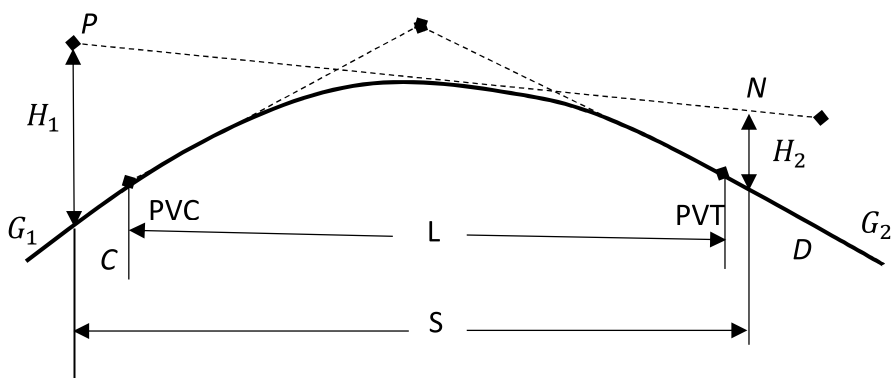
1.When SSD is greater than the length of the vertical curve.

$$L_{min} = 2SSD - \frac{200\left(\sqrt{H_1} + \sqrt{H_2}\right)^2}{A} \quad (for\ S > L)$$

Substituting 1.08m for H_1 and 0.60m for H_2

$$L_{min} = 2SSD - \frac{658}{A} \quad (for\ S > L)$$

Where A the algebraic difference in grades



Sight Distance on Crest Vertical Curve ($S > L$)

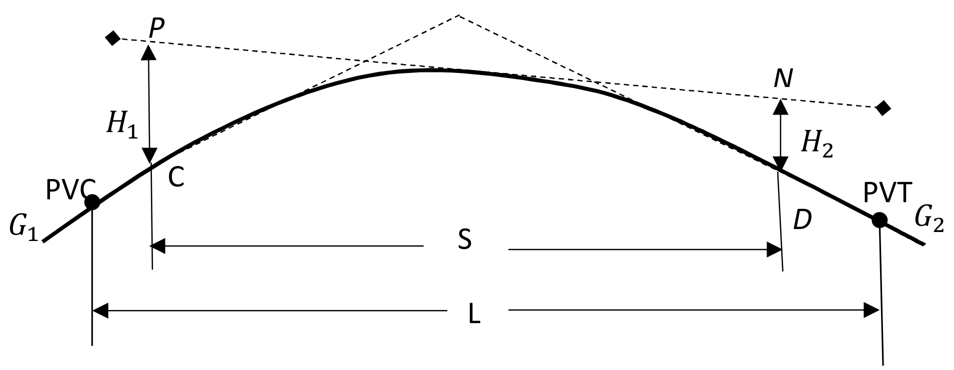
L = Length of vertical curve (m).
S = Sight distance (m).
H_1 = Height of eye above roadway surface (m).
H_2 = Height of object above roadway surface (m).
G_1, G_2 =grades of tangents (%)
PVC = point of vertical curve.
PVT = point of vertical tangent.
PN = driver's line of sight.

2. When SSD is less than the length of the vertical curve.

$$L_{min} = \frac{ASSD^2}{200\left(\sqrt{H_1} + \sqrt{H_2}\right)^2} \; (for\ S < L)$$

Substituting 1.08m for H_1 and 0.6m for H_2

$$L_{min} = \frac{ASSD^2}{658} (for\ S < L)$$



(Fig 15.13) Sight Distance on Crest Vertical Curve ($S < L$)

L = Length of vertical curve (m).
S = Sight distance (m).
H_1 = Height of eye above roadway surface (m).
H_2 = Height of object above roadway surface (m).
G_1, G_2 =grades of tangents (%)
PVC = point of vertical curve.
PVT = point of vertical tangent.

Example 15.1 Minimum Length of a Crest Curve

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 96km/h, and a perception-reaction time of 2.5 sec. The deceleration rate for braking (a) is 3.41 m/s^2.

Solution:

- Use the equation derived in chapter 3 to determine the SSD required for the design conditions. (Since the grade changes constantly on a vertical curve, the worst-case value for G of 3% is used to determine the braking distance.)

$$SSD = 0.278ut + \frac{u^2}{254\left\{\left(\frac{a}{9.81}\right) - G\right\}}$$

$$= 0.278 \times 96 \times 2.5 + \frac{96^2}{254\left\{\frac{3.41}{9.81} - 0.03\right\}}$$

$$= 66.72 + 114.24$$

$$= 180.96$$

- Use Eq. 15.5 to obtain the minimum length of vertical curve:

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

$$= \frac{5 \times (180.96)^2}{658}$$

$$= 248.83\ m$$

Example 15.2 Maximum Safe Speed on a Crest Vertical Curve.

An existing vertical curve on a highway joins a +4.4% grade with a -4.4% grade. If the length of the curve is 82m, what is the maximum safe speed on this curve? What speed should be posted if 8km/h increments are used? Assume $a = 3.41m/sec^2$, perception-reaction time = 2.5sec, and that $S < L$

Solution:

- Determine the SSD using the length of the curve and Eq. 15.5.

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

$$82 = \frac{8.8 \times S^2}{658}$$

$$S = 78.30m$$

- Determine the maximum safe speed for this sight distance using the equation for SSD.

$$78.30 = 0.278 \times 2.5u + \frac{u^2}{254\left\{\frac{3.41}{9.81} - 0.044\right\}}$$

Which yields the quadratic equation

$$u^2 + 53.6u - 6038 = 0$$

- Solve the quadratic equation to find the u, the maximum safe speed.

$$u = 55.4km/h$$

The maximum safe speed for an SSD of 78.30m is therefore 55.4km/h.
If a speed limit is to be posted to satisfy this condition, a conservative value of 48km/h will be used.

Table 3.4: Design Controls for Crest Vertical Curves Based on Passing-Sight Distance.

Design Speed (km/h)	Minimum Passing-Sight Distance for Design (m)	Rate of Vertical Curvature, K. Rounded for Design (Length (m) per % of A)
30	217	50
40	285	90
50	345	130
60	407	180
70	482	250
80	541	310
90	605	390
100	670	480
110	728	570
120	792	670

Passing-sight Distance and Crest Vertical Curve Design.

In addition to stopping-sight distance, in some instances it may be desirable to provide adequate passing-sight distance, which can be an important issue in two-lane highway design (one lane in each direction). Passing-sight distance is a factor only in crest curve design because, for sag curves, the sight distance is unobstructed looking up or down grade, and at night, the headlights of oncoming or opposing vehicles will be noticed.

$$L_m = 2PSD - \frac{946}{A} \qquad for\ PSD > L$$

$$L_m = \frac{A\ PSD^2}{946} \qquad for\ PSD\ < L$$

Example:

An equal-tangent crest vertical curve is 480m long and connects a +2.0% and a -1.5% grade. If the design speed of the roadway is 80 km/h, does this curve have adequate passing-sight distance?

Solution:
To determine the length of curve required to provide adequate passing-sight distance at a design speed of 80 km/h, we use L = KA with K = 310 (Table 3.4) This gives:

$$L = 310(3.5) = 1085m$$

Because the curve is only 480 m long it is not long enough to provide adequate passing-sight distance.

Length of Sag Vertical Curves

The selection of the minimum length of a sag vertical curve is controlled by the following four criteria:

1. SSD provided by the headlight.
2. Comfort while driving on the curve.
3. General appearance of the curve.
4. Adequate control of drainage at the low point of the curve.

The headlight criterion is the one commonly used for establishing minimum lengths of sag vertical curves.

Minimum Length of Sag Vertical Curve based on SSD Criterion

1 The SSD is greater than the length of the vertical curve.

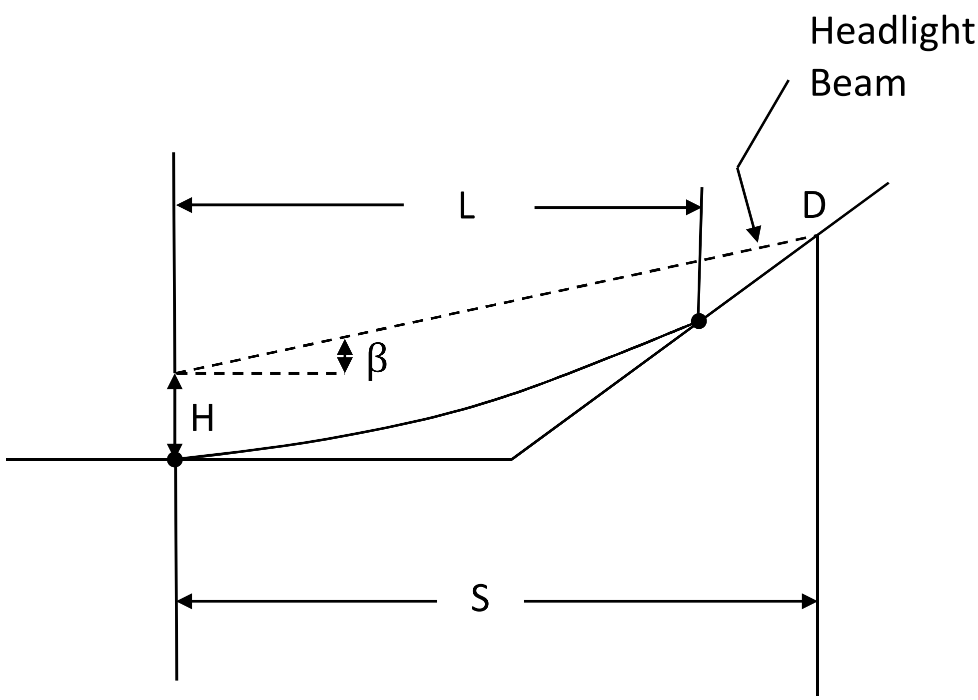
$$L_{min} = 2S - \frac{200(H+S\tan\beta)}{A} \ (for\ S>L) \qquad \textbf{(15.6)}$$

The values used by AASHTO for H and β are 0.60m (2ft) and 1 degree, respectively, so:

$$L = 2S - \left(\frac{120+3.5\,S}{A}\right) \ (Metric) \qquad \textbf{(15.7)}$$

Where H= the headlights above road level.

β = The upward inclination of the headlight beam to the horizontal



(Fig 15.14) Headlight Sight Distance on Sag Vertical Curves (*S >L)*

2 The SSD is less than the length of the vertical curve.

$$L_{min} = \frac{AS^2}{200\ (H + S\tan\beta)}\ (for\ S < L)$$

(15.8)

Substituting 0.60m (2ft) for H and 1 degree for β, yields:

$$L_{min} = \frac{AS^2}{120 + 3.5\ S}\ (for\ S < L) \quad (Metric)$$

(15.9)

To provide a safe condition, the minimum length of the sag vertical curve should assure a light beam sight distance *S* at least equal to the SSD. The SSD for the appropriate design speed is the value for *S* when the equations 15.8 and 15.9 are used to compute minimum lengths of sag vertical curves.

Minimum Length of Sag Vertical Curve based on Comfort Criterion

The comfort criterion is based on the fact that when a vehicle travels on a sag vertical curve, both the gravitational and centrifugal forces act in combination, resulting in a greater effect than on a crest vertical curve where these forces act in opposition to each other. Several factors such as weight carried, body suspension of the vehicle, and tire flexibility affect comfort due to change in vertical direction, making it difficult for comfort to be measured directly. It is generally accepted that a comfortable ride will be provided if the radical acceleration is not greater than 0.3048 m/s^2 ($1ft/sec^2$).

The following expression is used for the comfort criterion:

$$L_{min} = \frac{Av^2}{395} \quad (Metric.) \qquad \textbf{(15.10)}$$

Where:

v = is the design speed in km/h

L = the minimum length base on comfort in m.

A = the algebraic difference in grades.

The length obtained from the equation above is typically about 75 percent of that obtained from the headlight sight distance requirement.

Minimum Length of Curve based on Appearance Criterion

$$L_{min} = 30A \quad (Metric.) \qquad \textbf{(15.11)}$$

$$L_{min} = 60m \; if \;\; A < 2$$

Where L is the minimum length of the sag vertical curve in m or ft. Longer curves are frequently necessary for major arterials if the general appearance of these highways is to be considered to be satisfactory.

Minimum length of Curve based on Drainage Criterion.

`To satisfy the drainage criterion a minimum slope of 0.35 percent provided within 15m of the lowest point of the curve.

Example 15.3 Minimum Length of a Sag Vertical Curve

A sag vertical curve is to be designed to join a -5% grade to a +2% grade. If the design speed is 64 km/h, determine the minimum length of the curve that will satisfy all criteria.
Assume $a = 3.41\ m/sec$ and perception-reaction time = 2.5s.

Solution:

- Find the stopping distance.

$$SSD = 0.278ut + \frac{u^2}{254\left(\frac{3.41}{9.81} - G\right)}$$

$$= 0.278 \times 64 \times 2.5 + \frac{64^2}{254(0.35 - .05)} = 44.48 + 53.75$$

$$= 98.23m$$

- Determine whether $S < L\ or\ S > L$ for the headlight sight distance criterion.

$For\ S > L,$

$$L_{min} = 2S - \frac{(120 + 3.5S)}{A}$$

$$= 2 \times 98.23 - \frac{120 + 3.5 \times 98.23}{7}$$

$$= 130.2m$$

(This condition is not appropriate since $98.23\ < 130.20$.
Therefore $S \ngtr L$)

$For\ S < L,$

$$L_{min}\frac{AS^2}{120 + 3.5S}$$

$$= \frac{7 + (98.23)^2}{120 + 3.5 \times 98.23}$$

$$= 145.63m$$

This condition is satisfied since $98.23 \ < 145.63.$

- Determine minimum length for the comfort criterion.

$$L_{min=}\frac{Au^2}{395}$$

$$= \frac{7 \times 64^2}{395} = 72.60m$$

- Determine minimum length for the general appearance criterion.

$$L_{min} = 30A$$
$$= 30 \times 7 = 210m$$

The minimum length to satisfy all criteria is 210 m.

Length of Crest and Sag Vertical Curves Based on K Factors

For crest vertical curves

$$L_{min} = 2S - \frac{658}{A}\ (for\ S > L) \qquad \textbf{(15.3)}$$

$$L_{min} = \frac{AS^2}{658}\ (for\ S < L) \qquad \textbf{(15.5)}$$

Equation 15.5 can be written as:

$$L = KA \qquad \textbf{(15.12)}$$

Where K is the length of the vertical curve per percent change in A.
Since K is a function of design speed, it can be used as a convenient "short cut" to compute the minimum length for a crest vertical curve.

(Table 15.5) Values of K for Crest Vertical Curves Based on Stopping Sight Distance

Metric			
Design Speed (km/h)	Stopping Sight Distance (m)	Rate of Vertical Curvature K^a	
		Calculated	Design
20	20	0.6	1
30	35	1.9	2
40	50	3.8	4
50	65	6.4	7
60	85	11.0	11
70	105	16.8	17
80	130	25.7	26
90	160	38.9	39
100	185	52.0	52
110	220	73.6	74
120	250	95.0	95
130	285	123.4	124
[a] Rate of vertical curvature, k, is the length of curve per percent algebraic difference in intersecting grades (A). $K = L/A$			

For Sag Vertical Curves

The headlight sight distance requirement is used for design purposes, the expressions for the minimum lengths are given in the following equations:

$$L_{min} = 2S - \frac{(120+3.5S)}{A} \ (for\ S > L) \qquad \textbf{(15.7)}$$

$$L_{min} = \frac{AS^2}{120+3.5S} \ (for\ S < L) \qquad \textbf{(15.9)}$$

Equation 15.9 can be written as:

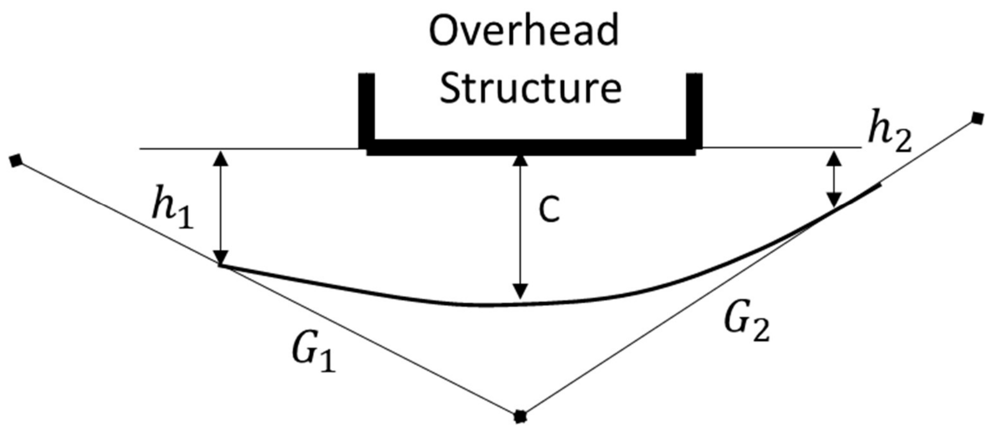
$$L_{min} = KA$$

The value of K for design where s is for a level road, are shown in table 15.6

Table 15.6 Values of K for Sag Vertical Curves Based on Stopping Sight Distance.

Metric			
Design Speed (km/h)	Stopping Sight Distance (m)	Rate of Vertical Curvature K^a	
		Calculated	Design
20	20	2.1	3
30	35	5.1	6
40	50	8.5	9
50	65	12.2	13
60	85	17.3	18
70	105	22.6	23
80	130	29.4	30
90	160	37.6	38
100	185	44.6	45
110	220	54.4	55
120	250	62.8	63
130	285	72.7	73
[a] Rate of vertical curvature, k, is the length of curve (m)per percent algebraic difference in intersecting grades (A). $K = L/A$			

Minimum Length of Sag Vertical Curve based on Sight Distance under Overhead Structure.



$$L_{min} = \frac{As^2}{800\left(C - \frac{H_1 + H_2}{2}\right)} \qquad S < L$$

$$L_{min} = 2S - \frac{800\left(C - \frac{H_1 + H_2}{2}\right)}{A} \qquad S > L$$

H_1 = Height for truck drivers eye level = 2.4m
H_2 = Height of tail light of a vehicle = 0.6m
C = Vertical clearance between road and overhead structure
= 5.2m if not given.

Elevation of Crest and Sag Vertical Curves

The method for computing elevations relies on the properties of the parabola. Consider a crest vertical curve as illustrated in figure 15.15 (A similar diagram if inverted would apply to a sag vertical curve.)
The equation of the curve, or tangent offset, is:

$$Y = ax^2$$

$$a = \frac{G_1 - G_2}{200L}\ (constant)$$

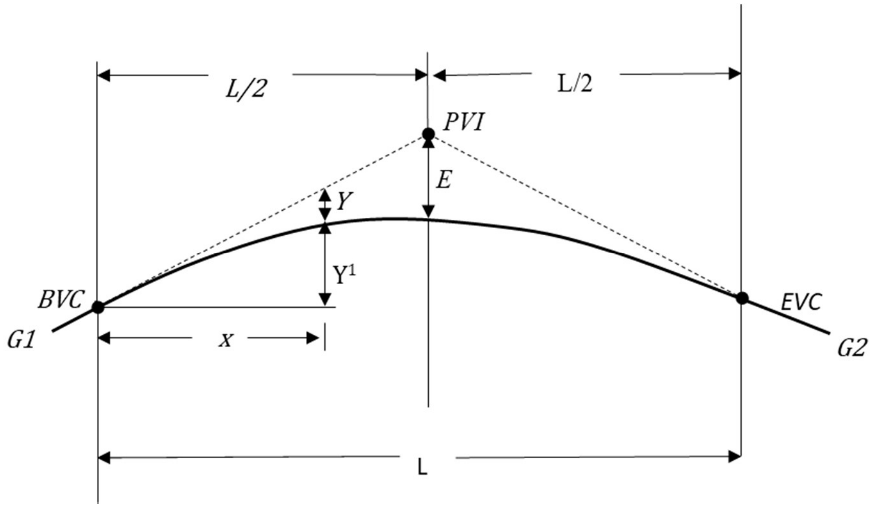
$$Y = \frac{A}{200}x^2 \tag{15.12}$$

Where $A = G_1 - G_2$

The external distance E from the point of vertical intersection (PVI) to the curve is determined by substituting $L/2$ for x in EQ 15.12

$$E = \frac{A}{200L}\left(\frac{L}{2}\right)^2 = \frac{AL}{800} \tag{15.13}$$

Figure 15.15 Layout of a Crest Vertical Curve for Design



PVI = Point of Vertical Intersection
BVC = Beginning of Vertical Curve (Same point as PVC)
EVC = End of Vertical Curve (Same point as PVT)
E=External Distance
G_1, G_2 = Grades of Tangents (%)
L = Length of Curve
A = Algebraic difference of grade $G_1 – G_2$

Since stations are given in 100ft intervals, E can be written as:

$$E = \frac{AN}{8}$$

(15.14)

Since stations are given in 30m intervals, E can be written as:

$$E = \frac{AN}{26.66}$$

Where N is the length of the curve in stations.
The vertical offset Y at any point on the curve can also be given in terms of E. Substituting 800 E/L for A in Eq. 15.12 will give:

$$y = \left(\frac{x}{L/2}\right)^2 E$$

(15.15)

The distance between the elevation at the beginning of the vertical curve (BVC) and the highest point on the curve, can be determined by considering the expression:

$$Y^1 = \frac{G_1 x}{100} - Y$$

$$= \frac{G_1 x}{100} - \frac{A}{200L} x^2$$

$$= \frac{G_1 x}{100} - \left(\frac{G_1 - G_2}{200L}\right) x^2$$

(15.16)

Differentiating EQ 15.16 and equating it to 0 will give the value of x at the highest point on the curve.

$$\frac{dY^1}{dx} = \frac{G_1}{100} - \left(\frac{G_1 - \ G_2}{100L}\right) x = 0$$

(15.17)

Therefore:

$$X_{high} = \frac{100\text{L}}{(G_1 - G_2)} \frac{G_1}{100} = \frac{\boldsymbol{LG_1}}{(\boldsymbol{G_1} - \boldsymbol{G_2})}$$

(15.18)

Where X_{High} = distance in feet from BVC to the turning point – that is, the point with the highest elevation on the curve. Equation 15.18 is also valid for sag curves.

Similarly, it can be shown that the difference in elevation between the BVC and the turning point $\left(Y_{high}^1\right)$ can be obtained by substituting the expression X_{high} for x in Eq. 15.16. The result, Eq. 15.19 is also valid for sag curves.

$$Y_{High}^1 = \frac{LG_1^2}{200(G_1 - G_2)}$$

(15.19)

<u>Design Procedure for Crest and Sag Vertical Curves</u>

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

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. 15.12) *as the distance between the tangent* and the curve. Usually equal distances of 100ft (1 Station) are used, beginning with the first whole station after the BVC.

Step 5 Compute elevations on the curve for each station as: elevation of the tangent ± offset from the tangent, Y. For crest curves the offset is (-) and for sag curves the offset is (+).

Step 6 Compute the location and elevation of the highest (Crest) or lowest (Sag) point on the curve using Eqs. 15.18 and 15.19.

Example 15.4
Design of Crest Vertical Curve.

A crest vertical curve joining a +3 percent and a -4 percent grade is to be designed for 120 km/h. If the tangents intersect at station (345 + 18) at an elevation of 76.2 determine the stations and elevations of the BVC and EVC . Also, calculate the elevations of intermediate points on the curve at the whole stations. A sketch of the curve is shown in figure 15.16

Solution: For a design speed of 120km/h, K = 93.5. From Table 15.5,

Minimum Length $93.5\ x\ [3-(-4)] = 654.5m$
Station of BVC $= (345+18) - \left(\frac{21+2}{2}\right) = 334+21$
Station of EVC $= (334+21) + (21+25) = 356+16$
Elevation of BVC $= 76.2 - \left(0.03\ x\ \frac{654.5}{2}\right) = 66.38$

The remainder of the computation is efficiently done using the format shown in table 15.7

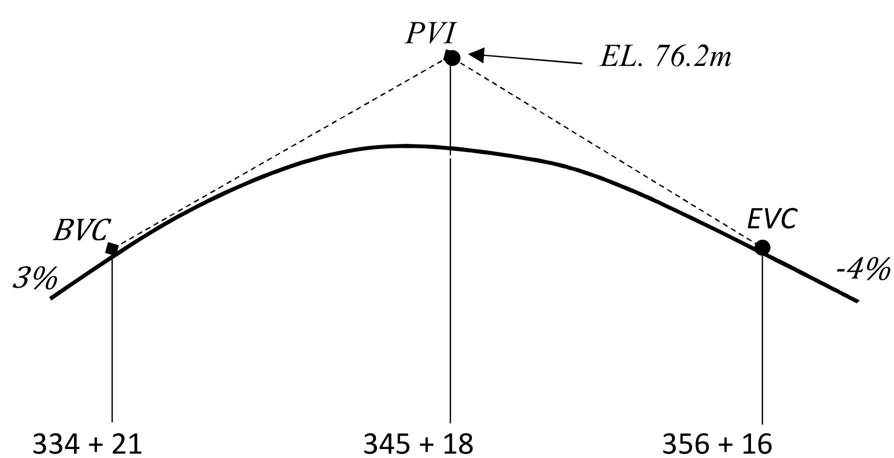
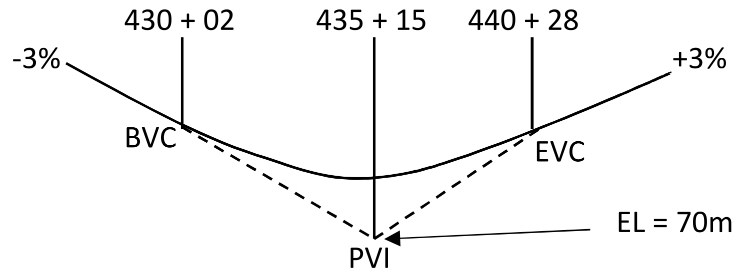


Table 15.7

Station	Distance from BVC $(x)\,(m)$	Tangent Elevation (m)	Offset $\left[y = \frac{Ax^2}{200L}\right](m)$	Curve Elevation (Tangent Elevation-Offset) (m)
BVC 334 + 21	0	66.38	0.00	66.38
BVC 335 + 00	10	$66.38 + \frac{10}{100} \times 3 = 66.68$	0.01	66.67
BVC 336 + 00	40	67.58	0.09	67.49
BVC 337 + 00	70	68.48	0.26	68.22
BVC 338 + 00	100	69.38	0.53	68.85
BVC 339 + 00	130	70.28	0.90	69.38
BVC 340 + 00	160	71.18	1.37	69.81
BVC 341 + 00	190	72.08	1.93	70.15
BVC 342 + 00	220	72.98	2.59	70.39
BVC 343 + 00	250	73.88	3.34	70.54
BVC 344 + 00	280	74.78	4.19	70.59
BVC 345 + 00	310	75.68	5.14	70.54
BVC 346 + 00	340	76.58	6.18	70.40
BVC 347 + 00	370	77.48	7.32	70.16
BVC 348 + 00	400	78.38	8.56	69.82
BVC 349 + 00	430	79.28	9.89	69.39
BVC 350 + 00	460	80.18	11.32	68.86
BVC 351 + 00	490	81.08	12.84	69.76
BVC 352 + 00	520	81.98	14.46	67.52
BVC 353 + 00	550	82.88	16.18	66.70
BVC 354 + 00	580	83.78	17.99	65.79
BVC 355 + 00	610	84.68	19.90	64.78
BVC 356 + 00	640	84.58	21.90	63.68
BVC 356 + 16	654.5	85.85	22.91	62.94

Example 15.15
Design of Sag Vertical Curve

A sag vertical curve joins a -3 percent grade and a +3 percent grade. If the PVI of the grades is at station (435+15) and has an elevation of 70m, determine the station and elevation of the BVC and EVC for a design speed of 112km/h. Also compute the elevation on the curve at 30 m intervals.
Figure 15.17 shows a layout of the curve.



Solution: For a design speed of 112 km/h, K = 54.3, using the higher rounded value in Table 15.6

Length of curve $= 54.3\ x\ 6 = 325.8$
Station of BVC $= (435 + 15) - (5 + 43) = 430 + 02$
Station of EVC $= (435 + 15) - (5 + 43) = 440 + 28$
Elevation of BVC $= 70 + 0.03\ x\ 163 = 74.89$
Elevation of BVC $= 70 + 0.03\ x\ 163 = 74.89$

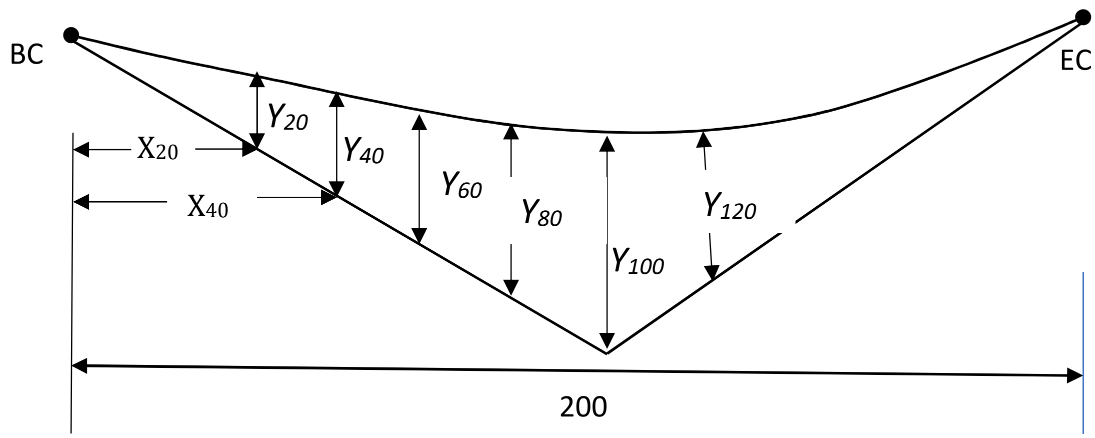
The computation of the elevations is shown in Table 15.8

Table 15.8 Elevation Computations for Example 15.5

Station	Distance from BVC (x) (m)	Tangent Elevation (m)	Offset $\left[y = \frac{Ax^2}{200L}\right] (m)$	Curve Elevation (Tangent Elevation-Offset) (m)
BVC 430 + 02	0	74.89	0.00	74.89
BVC 431 + 00	28	74.05	0.08	74.13
BVC 432 + 00	58	73.15	0.32	73.47
BVC 433 + 00	88	72.25	0.73	72.98
BVC 434 + 00	118	71.35	1.19	72.54
BVC 435 + 00	148	70.45	2.02	72.47
BVC 436 + 00	178	69.55	2.92	72.47
BVC 437 + 00	208	68.65	3.99	72.54
BVC 438 + 00	238	67.75	5.23	72.98
BVC 439 + 00	268	66.85	6.62	73.47
BVC 440 + 00	298	65.95	8.18	74.13
BVC 440 + 28	325.8	65.11	9.79	74.90

Example:

A -3% grade is connected to 1% by means of 200m vertical curve. If the elevation at the beginning of the curve is 100m, find the elevations of curve at 20m intervals.



$$Y = ax^2$$

$$a = \left|\frac{G_1 - G_2}{2L}\right|$$

$$= \frac{-3-(+1)}{2 \times 200} = \frac{-4\%}{400} = 0.0001$$

Hence:

$y_0 = ax^2$

$y_{20} = (0.0001)(20)^2 = 0.04$

$y_{40} = (0.0001)(40)^2 = 0.16$

$y_{60} = (0.0001)(60)^2 = 0.36$

$y_{80} = (0.0001)(80)^2 = 0.64$

$y_{100} = (0.0001)(100)^2 = 1$

$y_{120} = (0.0001)(80)^2 = 0.64$

$y_{140} = (0.0001)(60)^2 = 0.36$

$y_{160} = (0.0001)(40)^2 = 0.16$

$y_{180} = (0.0001)(20)^2 = 0.04$

$y_{200} = 0$

Station	Distance from BVC $(x)\,(m)$	Grade	Tangent Elevation	Offset	Profile Elevation
BVC	0+00	-3%	100	Zero	100
	20		99.40	0.04	99.44
	40		98.80	0.16	98.96
	60		98.20	0.36	98.56
	80		97.60	0.64	98.24
PI	100	+1%	97.00	1	98.00
	120		97.20	0.64	97.84
	140		97.40	0.36	97.76
	160		97.60	0.16	97.76
	180		97.80	0.04	97.84
EVC	200		98.00	0.00	98.00

Find the minimum distance from PVC to the lowest point on the curve.

$$x_m = \left|\frac{G_1}{G_2}\frac{L}{G_1}\right|$$

$$= \frac{3 \times 200}{1+3} = \frac{600}{4} = 150$$

Tangent Elevation @ $150 = Elev\ at\ PI - G_2 \times 50$

$$= 97 + \frac{1}{100} \times 50$$

$$= 97 + 0.5 = 97.5$$

Profile Elevation @ $150 = 97.5 + Y_{150}$

$$= 97.5 + 0.0001(50)^2$$

$$= 97.75$$

Horizontal Alignment

- The horizontal alignment consists of straight sections of the road (known as tangents) connected by curves. The curves are usually segments of circles, which have radii that will provide for a smooth flow of traffic.
- The design of the horizontal alignment entails:
    - Determination of the minimum radius.
    - Determination of the length of the curve.
    - Computation of the horizontal offsets from the tangents to the curve to facilitate locating the curve in the field.

- To avoid a sudden change from a tangent with infinite radius to a curve of finite radius, a curve with radii varying from infinite to the radius of the circular curve is placed between the circular curve and the tangent. Such a curve is known as a Spiral or Transition Curve.
- The wider the curve (bigger radius) the better and safer it is.
- The higher the design speed the larger the radius.

There are four types of horizontal curves:

1. Simple Curves.
2. Compound Curves.
3. Reversed Curves.
4. Transition Curves (Spiral).

Simple Curves

The curve is a segment of a circle with Radius R, which is discussed in chap 3 for the case when SSD is unobstructed. The relationship is shown to be:

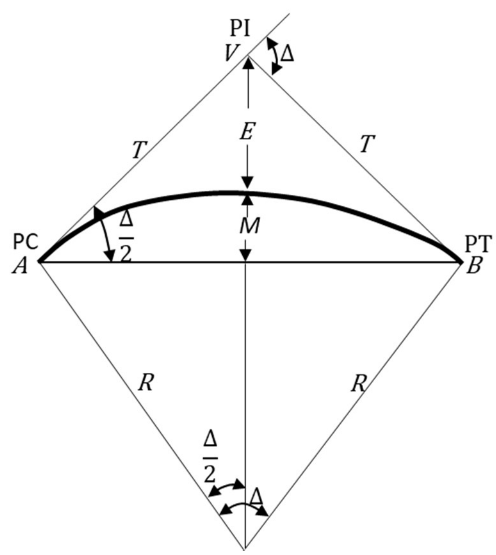
$$R = \frac{u^2}{g(e + f_s)}$$

Where:

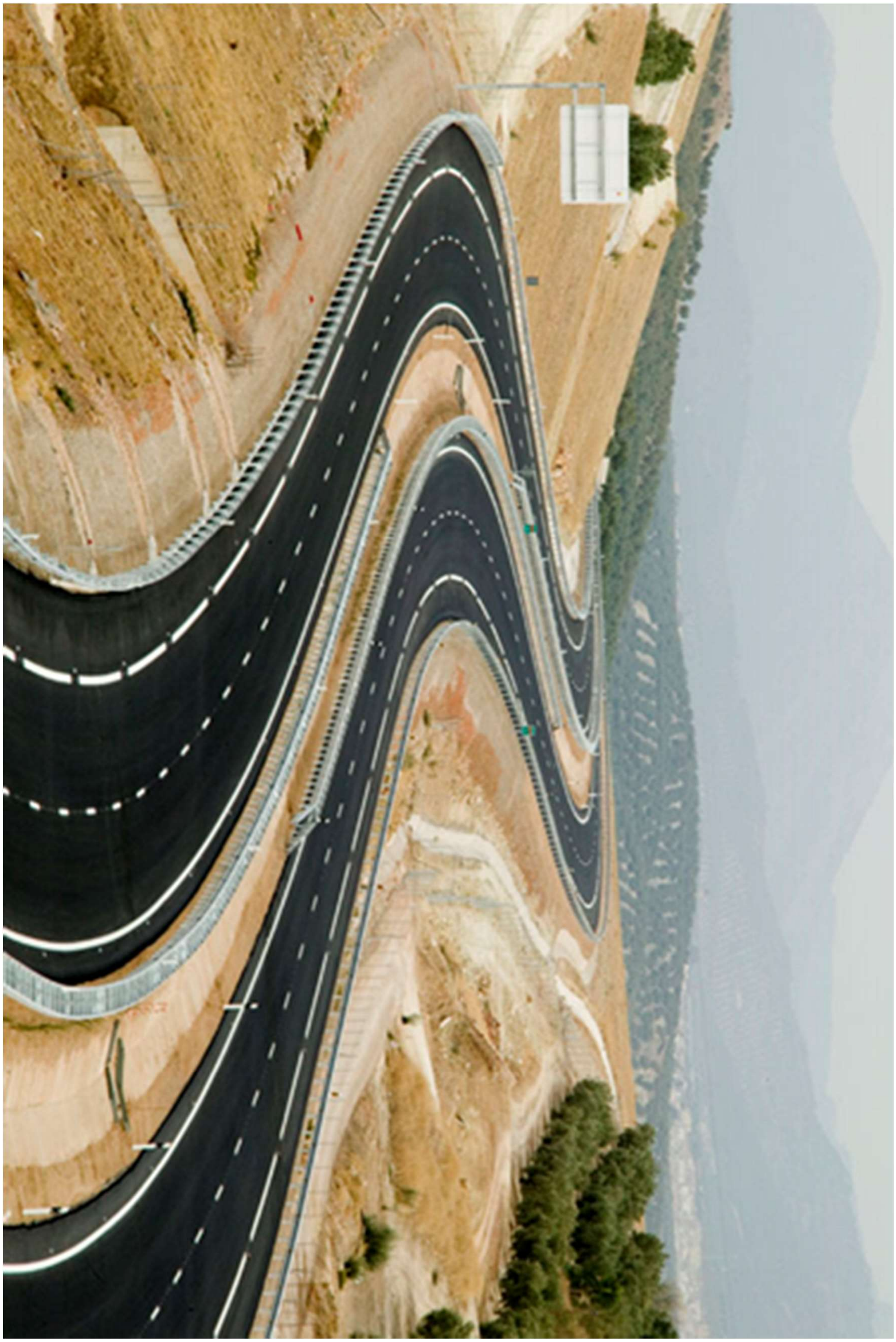
R = minimum radius (m.)
u = Design speed (km/h)
e = superelevation (m./m.)
f_s = coefficient of side friction

When there are obstructions in the roadway that limited SSD on the curve, R is determined as described later in this chapter.

Figure 15.18 Layout of a Simple Horizontal Curve

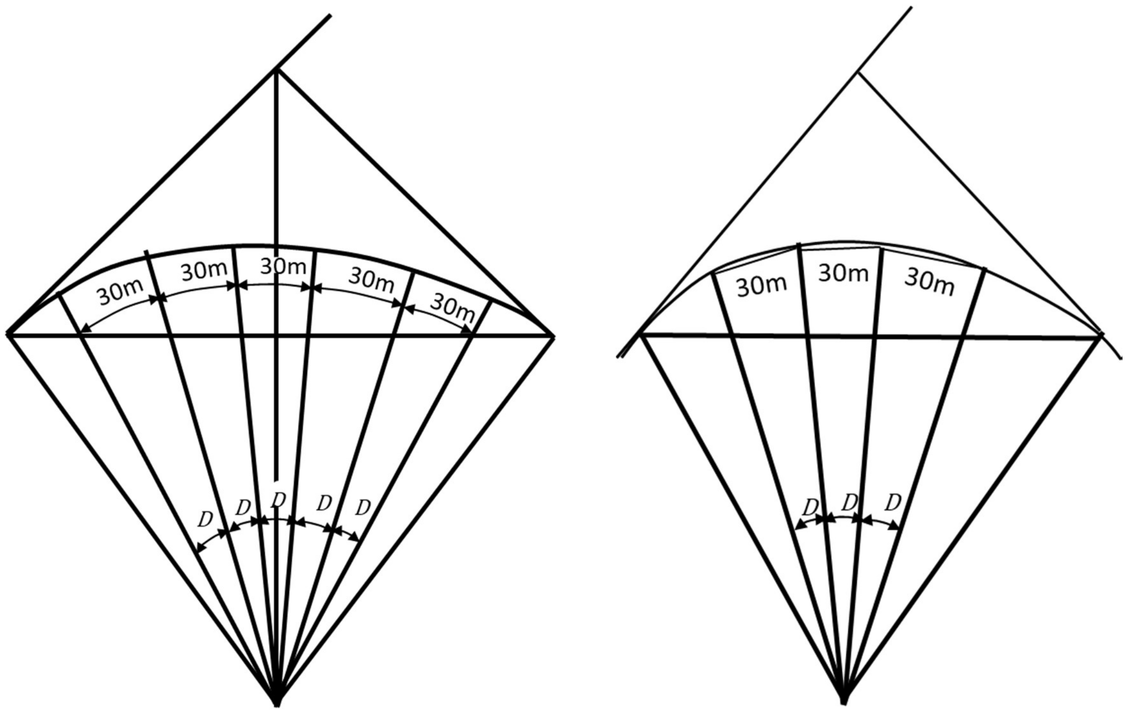


R = Radius of Circular Curve	PC = Point of Curve
T = Tangent Length	PT = Point of Tangent
Δ = Intersection Angle.	PI = Point of Intersection
M = Middle Ordinate	E = External Distance



Simple circular curve is described either by its radius, for example,60-m-radius curve, or by the degree of the curve, for example, a 4-degree curve. There are two ways to define degree of the curve, which is based on 30 m of arc length or on 30 m of chord length. Highway practice uses arc definition whereas railroad practice uses chord definition.

Figure 15.19 Arc and Chord Definitions for a Circular Curve



(a) Arc Definition

(b) Chord Definition

If ϴ is the angle in radians subtended at the center by an arc of a circle, the length of that arc is given by $R\theta = L$. If D_a° is the angle in degrees subtended at the center by an arc of length L, then:

$$\theta = \frac{\pi D_a^{\circ}}{180}$$

$$R = \frac{1719}{D_a^{\circ}} \quad (Metric) \qquad \textbf{(15.20)}$$

So the degree of curve can thus be determined if the radius is known, or the radius can be determined if the degree of curve is known.

<u>Chord Definition</u> for R in terms of D_c is based on a chord of 30 m as illustrated in figure 15.19 (b). For this case, the radius of curve is as follows.

$$R = \frac{15}{sin\ D_c^{\circ}/2} \qquad \textbf{(15.21)}$$

Since the arc definition is used for highway work, all further references to degree of curve in this chapter are to D_a.

Formulas for Simple Curves.

The expression for the tangent length is:

$$T = R\, tan\frac{\Delta}{2}$$

(15.22)

The length C of the chord AB, which is known as the long chord, is:

$$C = 2R\, sin\frac{\Delta}{2}$$

(15.23)

The expression for the external distance E, which is the distance from the point of intersection to the curve on a radial line is:

$$E = Rsec\frac{\Delta}{2} - R$$

$$E = R\left(\frac{1}{cos\frac{\Delta}{2}}\right) - 1$$

(15.24)

The expression for the middle ordinate M, which is the distance between the midpoint of the long chord and the midpoint of the curve is:

$$M = R - Rcos\frac{\Delta}{2}$$

$$= R\left(1 - cos\frac{\Delta}{2}\right)$$

(15.25)

The expression for the length of the curve L is:

$$L = \frac{R\Delta\pi}{180}$$

(15.26)

Field Location of a Simple Horizontal Curve.

Simple horizontal curves are usually located in the field by staking out points on the curve using angles measured from the tangent at the point of curve (PC) and the lengths of the chords joining consecutive whole stations. The angles are also called “deflection angles” because they are the angle that is “deflected” when the direction of the tangent changes direction to that of the chord.

$$\frac{l_1}{\delta_1} = \frac{L}{\Delta} = \frac{l_2}{\delta_2}$$

Chord: $Ap = 2R\sin\frac{\delta_1}{2}$ Deflection Angle: $VAp = \frac{\delta_1}{2}$

Chord: $pq = 2R\sin\frac{D_a}{2}$ Deflection Angle: $VAq = \frac{\delta_1+D}{2}$

Chord: $sB = 2R\sin\frac{\delta_2}{2}$ Deflection Angle: $VAB = \frac{\delta_1+D+D+D+\delta_2}{2} = \frac{\Delta}{2}$

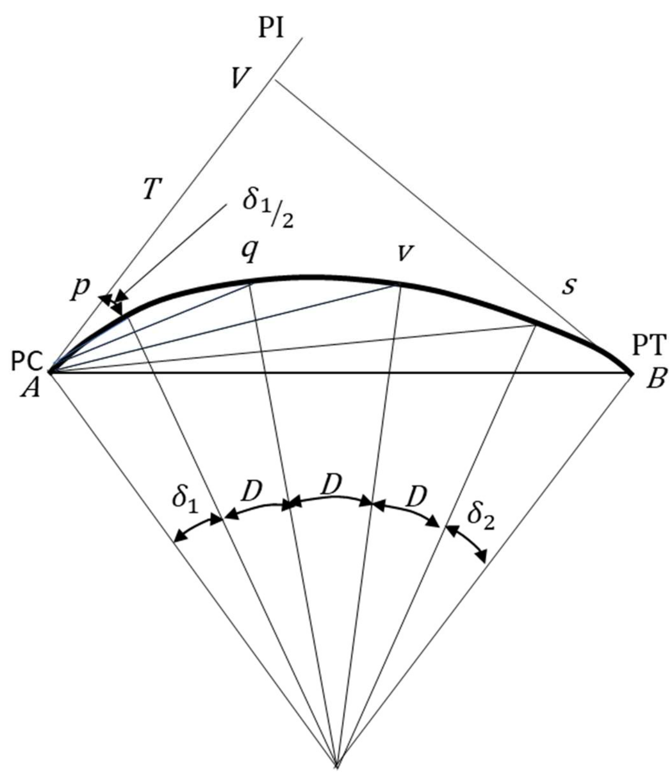


Figure 15.20 Deflection Angles on a Simple Curve

Example 15.6 Design of a Simple Horizontal Curve

The intersection angle of a 4° curve is 55°25’, and the PC is located at station 238 + 13.43. Determine the length of the curve, the station of the PT, the deflection angles and the chord lengths for setting out the curve at whole stations from the PC.
Figure 15.21 illustrates a layout of the curve.

Solution:

$$Radius\ of\ curve = \frac{1719}{D} = \frac{1719}{4}$$

$$\approx 429.8m.$$

$$Length\ of\ curve = \frac{R\Delta\pi}{180} = \frac{429.8 \times 55.4167\pi}{180}$$

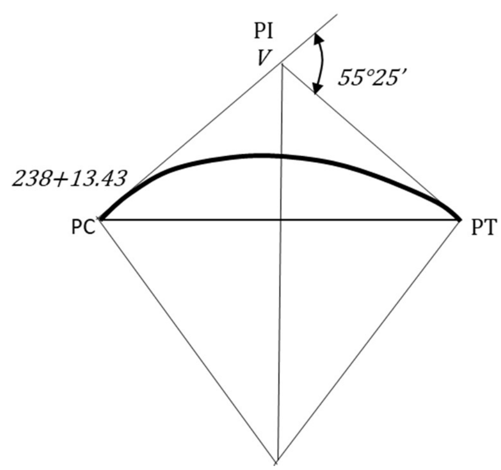
$$= 415.7m.$$

The station at PT is equal to station $(238 + 13.43) + (13 + 25.63) = 252 + 9.05$ stations. The distance between the PC and the first station is $239 - (238 + 13.43) = 16.57m$.

$$\frac{\delta_1}{\Delta} = \frac{l_1}{L}$$

$$\frac{\delta_1}{55.4167} = \frac{16.57}{415.7}$$

Figure 15.21 Layout of Curve for Example 15.6



Therefore:

$$\delta_1 = 2.210$$

$$C_1 = 2 \times 429.8\ sin\left(\frac{2.210}{2}\right) = 16.58m.$$

The first deflection angle to station 239 is $\delta_1/2$=1.105°=1°6'18".
Similarly,

$$l_2 = (252 + 9.05) - (252) = 9.05m.$$

$$\frac{\delta_2}{2} = \frac{9.05}{415.7} \times \frac{55.4167}{2} = 0.6034°$$

$$= 36'12"$$

$$C_2 = 2 \times 429.8\ sin(0.6034°)$$

$$= 9.05m.$$

$$D = 4°$$

$$C_D = 2 \times 429.8\ sin\left(\frac{4}{2}\right)$$

$$= 30m.$$

Note that the deflection angle to PT is half the intersection angle Δ of the tangents. This relationship serves as a check of the computation. Since highway curves are relatively flat, the chords length are approximately equal to the arc lengths. The other deflection angles are computed in table 15.9.

Table 15.9 Computations of Deflection Angles and Chord Lengths for Example 15.6

Station	Deflection Angle	Chord Length (m.) $C = 2R\, sin\,\frac{\delta_1}{2}$
PC 238+ 13.43	0	0
PC239	1°6’18”	16.58
PC240	3°6’18”	30
PC241	5°6’18”	30
PC242	7°6’18”	30
PC243	9°6’18”	30
PC 244	11°6’18”	30
PC 245	13°6’18”	30
PC 246	15°6’18”	30
PC 247	17°6’18”	30
PC 248	19°6’18”	30
PC 249	21°6’18”	30
PC 250	23°6’18”	30
PC 251	25°6’18”	30
PC 252	27°6’18”	30
PT 252+9.06	27°42’30”	9.05

Example:

Given a Horizontal curve with a radius of 410m a deflection angle of 32° and the PI Station of 1+120.744. Compute the curve data and the station of BC and EC. Compute the deflection angle @ even 20m stations.

Solution:

The curve data.

$$T = R\tan\frac{\Delta = 410\, tan}{2}\frac{32}{2} = 117.56m$$

$$C = 2Rsin\frac{\Delta}{2} = 2 \times 410\ sin\frac{32}{2} = 2226.02m$$

$$M = R - Rcos\frac{\Delta}{2} = 410 - 410\ cos\frac{32}{2} = 15.88m$$

$$E = \frac{R}{cos\frac{\Delta}{2}} - R = \frac{410}{cos16} - 410 = 16.5m$$

$$L = R\Delta_{rad} = 410 \times \frac{32 \times \pi}{180} = 228.98m$$

Station of BC = Station of PI – T

$$1 + 120.744 - 117.56$$

$$1 + 003.18$$

Station of EC = Station of BC + L

$$= 1 + 003.18 + 228.98$$

$$= 1 + 232.6$$

Deflection Angle @ even 20m Intervals

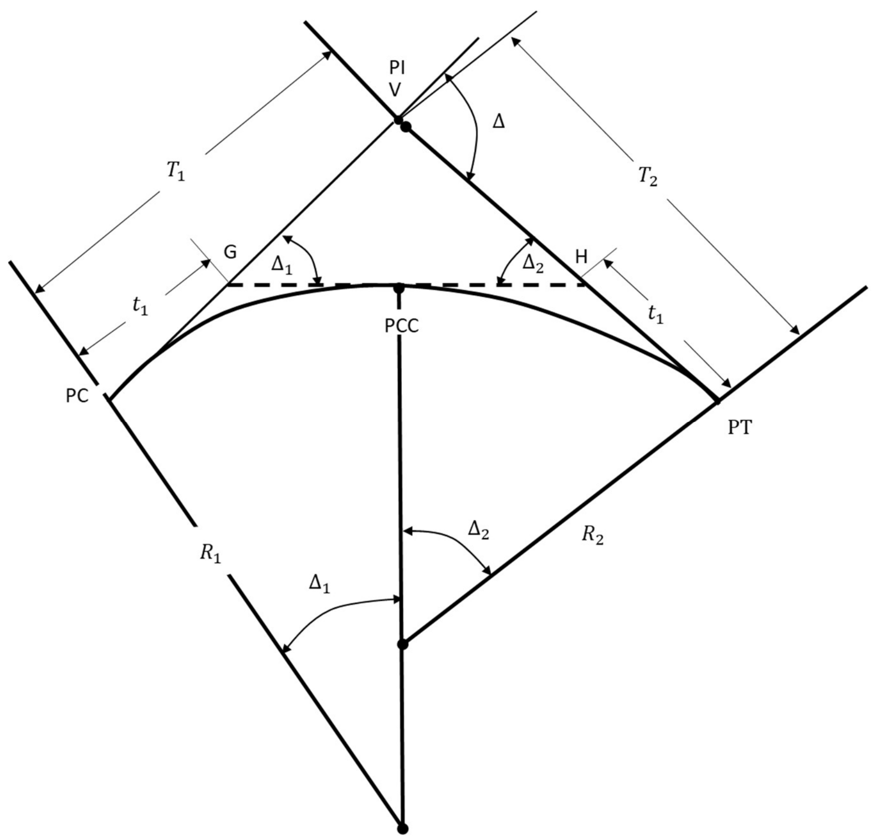
Station	Arch Length	Deflection Angle $\left(\frac{x}{2R}\right) \times \frac{180}{\pi}$	Chord Length $C = 2R\ sin\ d_x(m)$
BC 1+003.2	0	0	0
BC 1+023.2	20	1.39	19.89
BC 1+043.2	40	2.79	39.90
BC 1+063.2	60	4.19	59.99
BC 1+083.2	80	5.59	79.87
BC 1+103.2	100	6.99	99.75
BC 1+123.2	120	8.39	119.6
BC 1+143.2	140	9.78	139.29
BC 1+163.2	160	11.18	158.99
BC 1+183.2	180	12.58	178.59
BC 1+203.2	200	13.98	198.09
BC 1+223.2	220	15.38	217.48
BC 1+232.2	229	16.0	226.02

Compound Curves

Compound curves consist of two or more simple curves in succession, turning in the same direction, with any two successive curves having a common tangent point.

AASHTO recommends that the ratio of the flatter radius to the sharper radius at intersections should not be greater than 2:1 so drivers can adjust to sudden changes in curvatures and speed. The maximum desirable Ratio recommended for interchanges is 1.75:1, although 2:1 may be used.

Figure 15.22 Layout of a Compound Curve.



R_1, R_2 = radii of simple curves forming compound curve.

Δ_1, Δ_2 = Intersection angles of simple curves.

Δ = Intersection angles of compound curves.

t_1, t_2 = Tangent lengths of simple curves.

T_1, T_2 = Tangent lengths of compound curves.

PCC = Point of compound curve.

PI = Point of intersection.

PC = Point of curve.

PT = point of tangent.

To provide a smooth transition from a flat curve to a sharp curve and to facilitate a reasonable deceleration rate on a series of curves of decreasing radii. The length of each curve should observe minimum length requirements based on the radius of each curve as recommended by AASHTO. And given in table 15.10.

Values in table 15.10 are developed on the premises that travel is in the direction of the sharper curve. The 2:1 ratio of the flatter radius should preferably not be exceeded but is not critical for the acceleration condition.

Table 15.10 Lengths of Circular Arc for Different Compound Curve Radii

	Radius (ft.)						
Minimum Length of Circular Arc (ft.)	100	150	200	250	300	400	500 or more
Acceptable	40	50	60	80	100	120	140
Desirable	60	70	90	120	140	180	200

In figure 15.22 R_1 and R_2 are usually known. The following equations can be used to determine the remaining valuables.

$$\Delta + \Delta_1 + \Delta_2 \tag{15.29}$$

$$t_1 = R_1 \tan\frac{\Delta_1}{2} \tag{15.30}$$

$$t_2 = R_2 \tan\frac{\Delta_2}{2} \tag{15.31}$$

$$\frac{\overline{VG}}{\sin\Delta_2} = \frac{\overline{VH}}{\sin\Delta_1} = \frac{t_1 + t_2}{\sin(180 - \Delta)} = \frac{t_1 + t_2}{sin\,\Delta} \tag{15.32}$$

$$T_1 = \overline{VG} + t_1 \tag{15.33}$$

$$T_2 = \overline{VG} + t_2 \tag{15.34}$$

In order to lay out the curve, the **intersection angles and chord lengths for both curves must be determined**. Usually, Δ_1 or Δ_2 can be obtained from the layout plans and equations 15.29 through 15.34 can be used to solve for Δ_1 or Δ_2, VG, VH, t_1, t_2, T_1 and T_2 .

Example 15.7 Design of a Compound Curve

Figure 15.23 illustrates a compound curve that is to be set out at a highway intersection. If the point of compound curve (PCC) is located at station (565+10.5), determine the deflection angles for setting out the curve.

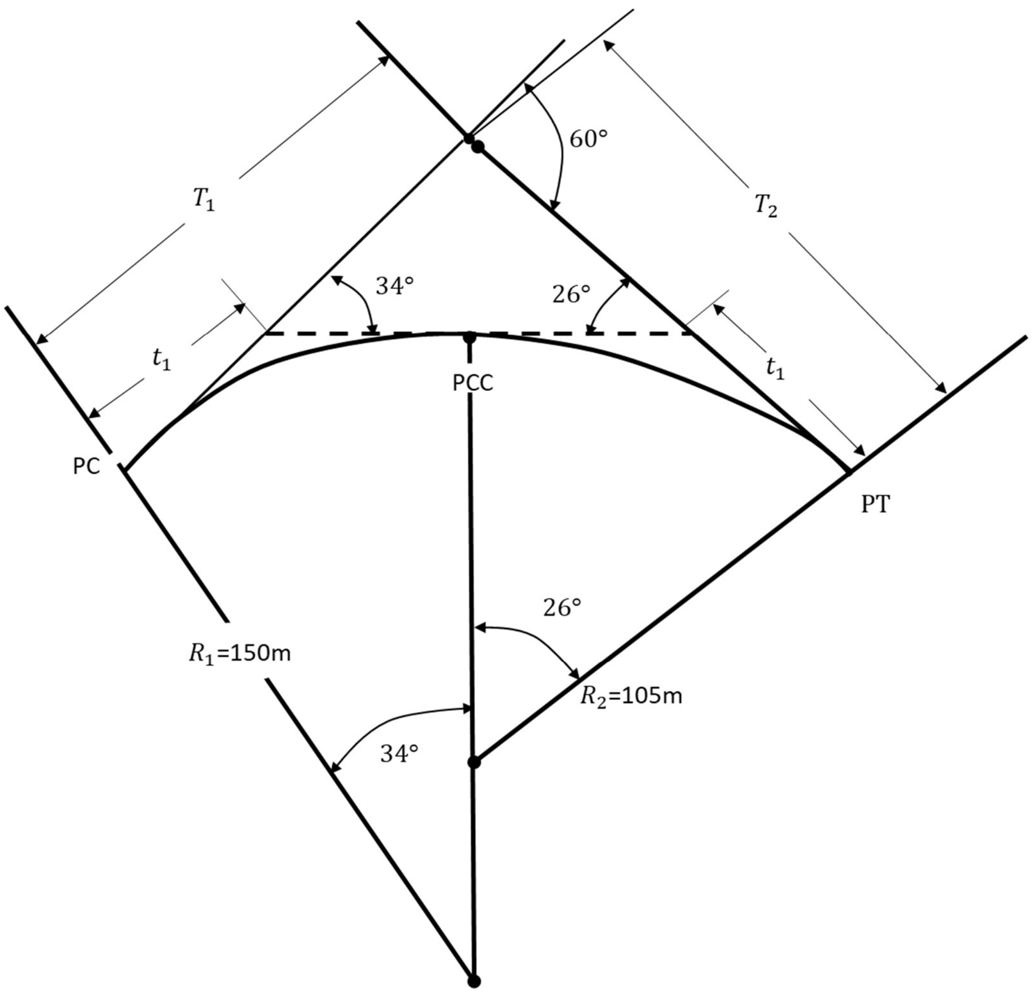


Figure 15.23 Compound Curve for Example 15.7

Solution:

$$t_1 = 150 \tan\frac{34}{2} = 45.86m$$

$$t_2 = 105 \tan\frac{26}{2} = 24.24m$$

For length of horizontal curve of 150m radius,

$$L = R\Delta_1 \frac{\pi}{180} = 150 \times \frac{34\pi}{180}$$

$$= 89.01m$$

For length of horizontal curve of 105m radius,

$$L = R\Delta_2 \frac{\pi}{180} = 105 \times \frac{26\pi}{180}$$

$$= 47.65m$$

Therefore:

Station of the PC is equal to $(565 + 10.50) - (2 + 29.01)$ =562+11.49

Station of PT is equal to $(565 + 10.50) + (1 + 17.65) = 566 + 28.15$.

For curve of 150m radius:

$$\frac{D}{2} = \frac{1719}{2 \times 150} = 5°43'47" \qquad (from\ Eq. 15.20)$$

$$l_1 = (563 + 00) - (562 + 11.49) = 18.51m$$

$$\frac{\delta_1}{l_1} = \frac{\Delta}{L}$$

$$\frac{\delta_1}{2} = \frac{18.51 \times 34}{2 \times 89.01} = 3°32'8"$$

$$l_2 = (565 + 10.50) - (565 + 00) = 10.5(m)$$

$$\frac{\delta_2}{2} = \frac{10.5 \times 34}{2 \times 89.01} = 2°0'19"$$

For curve of 105m radius,

$$\frac{D}{2} = \frac{1719}{2 \times 105} = 8°11'7"$$

$$l_1 = (566 + 00) - (565 + 10.50) = 19.5(m)$$

$$\frac{\delta_1}{2} = \frac{19.5 \times 26}{2 \times 47.65} = 5°19'16"$$

$$l_2 = (566 + 28.15) - (566 + 00) = 28.15$$

$$\frac{\delta_2}{2} = \frac{28.15 \times 26}{2 \times 47.65} = 7°40'44"$$

Computation of the deflection angles are in Table 15.11.
The deflection angles for the 105m radius curve are turned from the common tangent with the transit located at PCC. Since each simple curve is relatively flat, calculated lengths of the chords are almost equal to the corresponding arc lengths.

Table 15.11 Computations for Example 15.7

Station	150m Radius Curve Deflection Angle	Chord Length (m)
PC 562 + 11.49	0	0
563	3°32’8”	18.68
564	9°15’55”	48.3
565	14°59’42”	77.6
PCC 565 + 10.50	17°00’00”	87.7

Station	105 m Radius Curve	Chord Length (m)

	Deflection Angle	
PC 565 + 10.50	0	0
566	5°19’16”	19.47
PT 566+28.15	13°00’00”	47.24

Reverse Curves

Reverse curves usually consist of *two simple curve*s with equal radii turning in opposite directions with a common tangent. They are generally used to change the alignment of a highway.

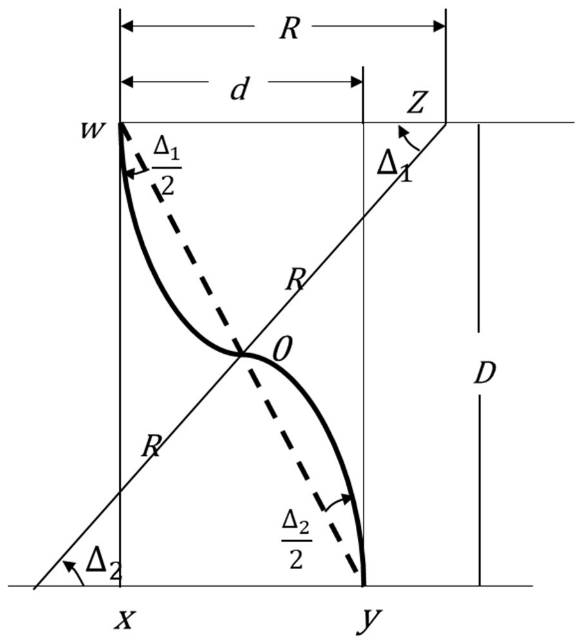


Figure 15.24 Geometry of a Reverse Curve with Parallel Tangents

R = Radius of simple curves.
Δ_1 Δ_2= Intersection angles of simple curves.
d = distance between parallel tangents.
D = distance between tangent points.

If d and D are known,
The following variables can be determined:

$$\Delta = \Delta_1 = \Delta_2$$

$$Angle\ OWX = \frac{\Delta_1}{2} = \frac{\Delta_2}{2}$$

$$Angle\ OYZ = \frac{\Delta_1}{2} = \frac{\Delta_2}{2}$$

Therefore, *WOY* is a straight line, and hence,

$$tan\frac{\Delta}{2} = \frac{d}{D}$$

$$d = R - Rcos\Delta_1 + R - Rcos\Delta_2$$

$$= 2R(1 - cos\Delta)$$

$$R = \frac{d}{2(1 - cos\Delta)}$$

(15.35)

If d and R are known,

$$cos\Delta = 1 - \frac{d}{2R}$$

$$D = d\ cos\frac{\Delta}{2}$$

Reverse curves are seldom recommended because sudden changes to the alignment may result in drivers finding it difficult to keep in their lanes. When it is necessary to reverse the alignment, a preferable design consists of *two simple horizontal curves, separated by a sufficient length of tangent* between them, to achieve superelevation. Alternatively, the simple curves may be separated by an equivalent length of spiral, which is described in the next section.

Spiral transition curves.

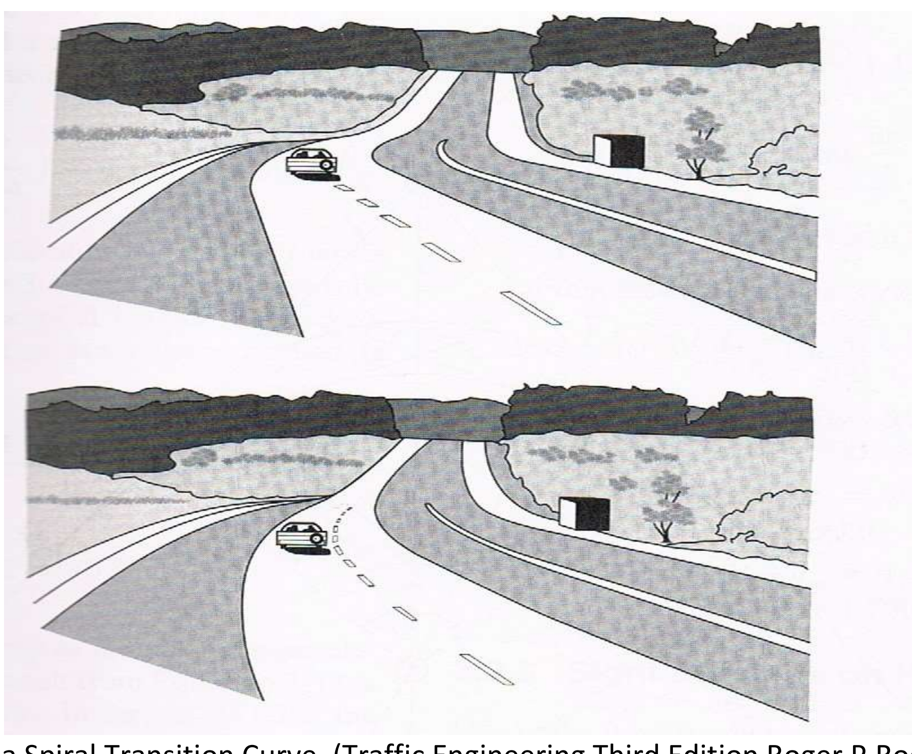
Transition curves are used in railways and in most highways. It is placed between the straight section and the circular curve. The radius of a transition curve varies from infinity at its tangent point with the straight to a minimum value at its tangent point with a circular curve.

A properly designed transition curve provides the following advantages;

- A natural, easy-to-flow path for drivers such that the centrifugal force increases and decreases gradually as a vehicle enters and leaves a circular curve.
- A convenient desirable arrangement for superelevation runoff.
- Flexibility in the widening of sharp curves.
- Enhancement in the appearance of the highway.

Spiral transition curves are not always used, as construction is difficult and construction costs are generally higher than for a simple circular curve.

When transition curves are not provided, driver tend to create their own transition curves by moving laterally within their travel lane and sometimes the adjoining lane, which is risky not only for them but also for other road users.



The visual Impact of a Spiral Transition Curve. (Traffic Engineering Third Edition Roger.P.Roess)

Length of Spiral Curve.

The expression given in Eqs. 15.37 and 15.38 is used by some highway agencies to compute the minimum length of a spiral transition curve. The minimum length should be the larger of the values obtained from these equations. The factor C is an empirical value representing the comfort and safety levels provided by the spiral curve. The value of $C = 0.3m/S^3$ is generally accepted for railroad operation, but values ranging from 0.3 to $0.9 m/S^3$ have been used for highway

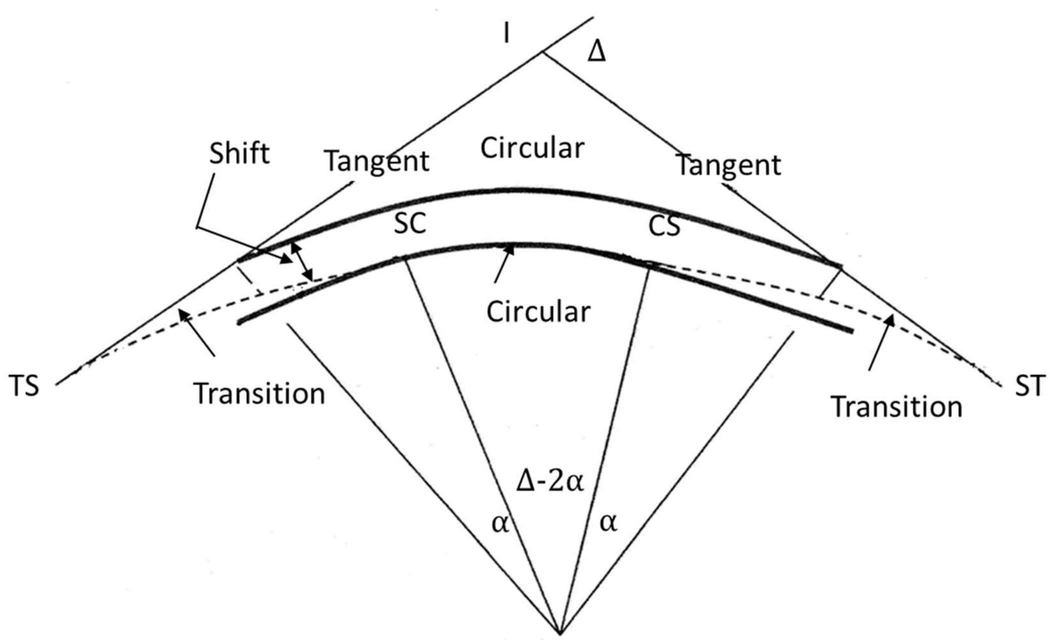
$$L_{s,min} = \frac{0.0215u^3}{RC}$$

(15.37)

$$L_{s,min} = \sqrt{24(p_{min})R}$$

(15.38)

$$\alpha = \frac{L_s}{2R}$$



Where:

L_s = Minimum Length of Spiral Curve (m)

u = Speed (km/h)

R = Radius of Circular Curve (m)

C = Rate of increase of radial acceleration m/s^3. Values range from 0.3 to 0.9 m/s^3.

P_{min} = Minimum Lateral Offset between the tangent and the circular curve (0.2m)

α = The central angle of the spiral curve (rad)

Example:

In order to improve an existing circular curve $R = 250m, \Delta = 31°20$, a transition curve is introduced. Determine the minimum length of this curve. The design speed = 80km/hr.

$$L_{s,min} = \sqrt{24R\ P_{min}} \qquad L_{s,min} = \sqrt{24 \times 250 \times 0.2} = 34.64m$$

$$L_{s,min} = \frac{0.0215V^3}{RC} \qquad L_{s,min} = \frac{0.0215 \times 80^3}{250 \times 0.9} = 48.7m$$

$$Use\ L_{s,min} = 48.7m$$

Example:

Two tangents intersect at a deflection angle = 35° at station I = 23 + 20. These tangents are to be connected by two similar transition curves 75m long and a circular curve, R = 300m. Calculate stations **TS, SC, CS, ST**

$$\text{A central angle of the spiral (rad)} = \frac{L_s}{2\ R} = \frac{75}{2 \times 300} = 0.125rad = 7°09'56"$$

35 -2α (central angle of the circular curve) = 20° 40’ 08”

Length of the circular curve $L = 20°42' \times \frac{\pi}{180} \times 300 = 108.16m$

$$Shift = \frac{L_s^2}{24R} = \frac{75^2}{24 \times 300} = 0.78m$$

$$Tangent\ Length = \frac{75}{2} + (300 + 0.78)tan\frac{35°}{2} = 132.34m$$

$$Station\ TS = Station\ I - tangent = 19 + 7.66$$
$$Station\ SC = Station\ TS + L_s = 21 + 22.66$$
$$Station\ CS = Station\ SC + L_{circular} = 25 + 10.80$$
$$Station\ ST = Station\ CS + L_s = 27 + 25.80$$

Desirable Length of Spiral

Desirable Length of Spiral transition curves are shown below correspond to 2.0s of travel time at the design speed of the roadway. This travel time has been found to be representative of the natural spiral path for most drivers. The spiral lengths listed in the table below are recommended as desirable values for street and highway design.

Metric	
Design Speed (km/h)	Spiral Length (m)
20	11
30	17
40	22
50	28
60	33
70	39
80	44
90	50
100	56
110	61
120	67
130	72

Desirable Length of Spiral Curve Transition

If the desirable spiral curve length shown in the table above is less than the minimum spiral length determined from equations (15.37) and (15-38), the minimum spiral curve length should be used in design.

Superelevation

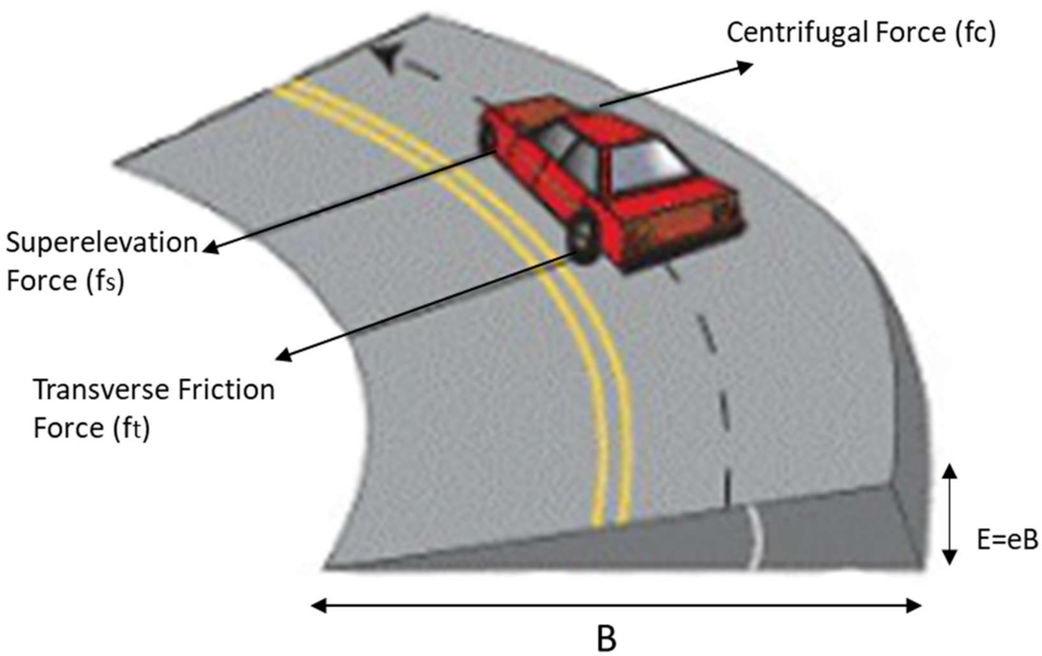
The purpose of superelevation of a curve is to counteract the centrifugal force produced as a vehicle travels round a curve.
Vehicles moving around a circular curve is subjected to two main forces.

1 Centripetal force $P = \frac{Wu^2}{gR}$ acting horizontally outwards.

2 The friction developed between the wheel and the pavement (f)

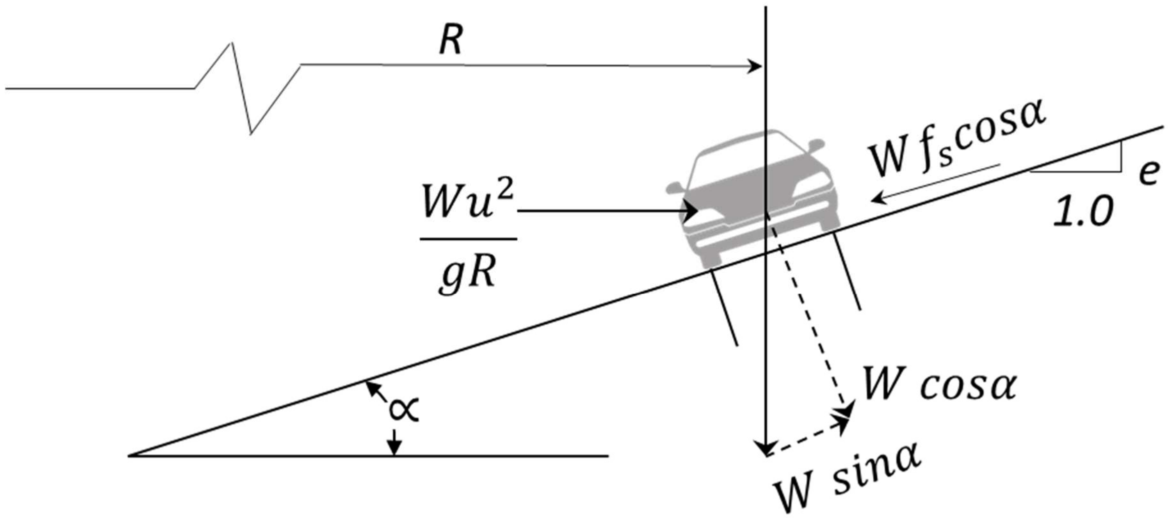
At high speed the friction force is not adequate to counterbalance the centripetal force. Therefore, the road must be inclined towards the center of the curve to provide an additional force to counterbalance the outward radial force. The inclination is known as Superelevation.

Maximum Superelevation.
Maximum superelevation rate is 4% for urban areas and 10 - 12% for rural areas without ice or snow.



Superelevation

Figure 3.8 Forces Acting on a vehicle Travelling on a Horizontal Curve Section of a Road.



W = Weight of vehicle.
f_s = Coefficient of side friction.
G = Acceleration of gravity.
U = Speed when brakes applied.
R = Radius of Curve.
α= Angle of incline.
e = tan α (rate of superelevation).
T = Track width.
H = Height center of gravity.

When the vehicle is in equilibrium with respect to the incline (that is, the vehicle moves forward but neither up nor down the incline), we may equate the three relevant forces and obtain;

$$\frac{Wu^2}{gR}\cos\alpha = W\sin\alpha + Wf_s\cos\alpha$$

(3.31)

Where f_s = coefficient of side friction and $\left(\frac{u^2}{g}\right) = R(tan\ \alpha + \ f_s)$. This gives;

$$R = \frac{u^2}{g(\tan\alpha + f_s)}$$

(3.32)

Tan α, the tangent of the angle of inclination of the roadway, is known as the rate of superelevation e. Equation 3.32 can therefore be written as:

$$R = \frac{u^2}{g(e + f_s)}$$

(3.33)

Again, if g is taken as 9.81 m/sec^2 and u is measured in km/h, the minimum radius R is given in m as;

$$R = \frac{u^2}{127\ (e + f_s)}$$

(3.34)

Equation 3.34 shows that to reduce R for a given velocity either e or f_s or both should be increased.

Equation 3.34 can be written as:

$$e + f_s = \frac{u^2}{127R}$$

Values of Side Friction recommended by ASSHTO

Design Speed (km/h)	Maximum Side Friction Factor
30	0.17
40	0.17
50	0.16
60	0.15
70	0.14
80	0.14
90	0.13
100	0.12
110	0.11
120	0.09

Example:

1) Find the rate of superelevation on a horizontal curve having a radius of curvature of 90m. The design speed 50km/h Assume f_s= 0.15

Solution:

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

$$e + 0.15 = \frac{50^2}{127 \times 90}$$

$$e = 0.22 - 0.15 = 0.07 \; or \; 7\%$$

2) If the road width is 7m calculate the rise or banking due to superelevation.

Solution:

If the width of road = B

Then: Total rise at the outer edge E

$$E = e + B$$

$$= 0.07 \times 7 = 0.49m$$

Achieving Superelevation.

The transition from a tangent section with a normal superelevation for drainage to a super-elevated horizontal curve occurs in two stages:

- **Tangent Runoff**: The outside lane of the curve must have a transition from the normal drainage superelevation to a level or flat condition prior to being rotated to the full superelevation for the horizontal curve. The length of this transition is called the tangent run off and is noted as $L_{t.}$
- **Superelevation Runoff:** Once a flat cross-section is achieved for the outside lane of the curve, it must be rotated (with the other lanes) to the full superelevation rate of the horizontal curve. The length of this transition is called the superelevation runoff and is noted as L_s

For most undivided highways, rotation is around the centerline of the roadway, although rotation can also be accomplished around the inside or outside edge of the roadway as well. For divided highways, each directional roadway is separately rotated, usually around the inside or outside edge of the roadway.
Figure 3.9 illustrates the rotation of undivided two-lane and four-lane is rotated highway around the center-line, although the slopes shown are exaggerated for clarity. The rotation is accomplished in three steps:

1) The outside lane(s) are rotated from their normal cross-slope to a flat condition.
2) The outside lane(s) are rotated from the flat position until they equal the normal cross-slope of the inside lanes.
3) All lanes are rotated from the condition of step 2 to the full superelevation of the horizontal curve.

The tangent runoff is the distance taken to accomplish step 1, while the superelevation runoff is the distance taken to accomplish step 2 and 3. The tangent and superelevation runoffs are, of course, implemented for the transition from tangent to horizontal curve and for the reverse transition from horizontal curve back to tangent.

In effect, the transition from a normal cross-slope to a fully super-elevated section. Is accomplished by creating a grade differential between the rotation axis and the pavement edge lines. To achieve safe and comfortable operations, there are limitations on how much of a differential may be accommodated.

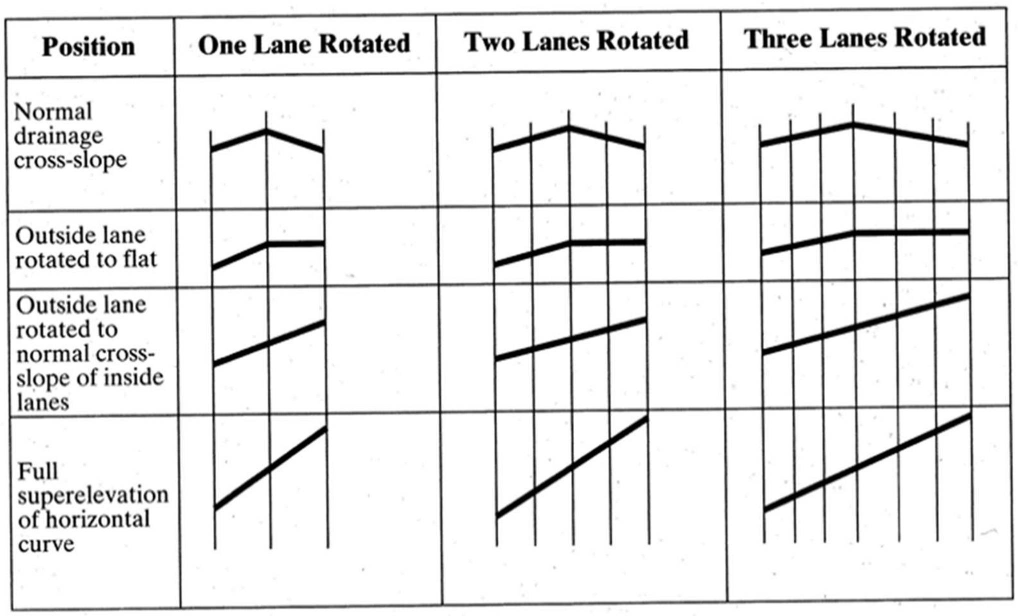


Figure 3.9 Achieving Superelevation by Rotation Around Center Line

Curve Radii Based on Stopping Sight Distance

It was shown in chapter 3 that the minimum radius of a horizontal curve depends on the design speed u of the highway, the superelevation e and the coefficient of side friction f_s. The minimum radius R is given as:

$$R = \frac{u^2}{g(e + f_s)}$$

$$R = \frac{u^2}{127(e + f_s)}$$

Where:

u = speed (km/h)

R = Radius of Curve (m)

f_s =coefficient of side friction

e = tan α (rate of Superelevation)

g = (9.81m/s^2)

Normally, this value for R is sufficient for design purposes. However, there are instances when a constraint may exist. For example, if an object is located near the inside edge of the road, the driver's view may be blocked. When this situation exists, the design of the horizontal curve based on design speed is compromised and the designer has two options:

(1) Change the Radius of curve to assure adequate SSD

(2) Post a lower speed on the curve.

If a vehicle is located at point A on the curve and the object is at point B, the line of sight is the length of chord AB. The horizontal distance traversed by the vehicle when moving from point A to point B is the length of arc AB. The central angle for arc AB is defined as 2θ (in degrees). Thus, the expression for SSD is.

$$\frac{2R\theta\pi}{180} = S$$

$$\theta = \frac{180(S)}{2\pi R} = \frac{28.65}{R}(S)$$

$$\frac{R-m}{R} = \cos\theta$$

$$\cos = \frac{28.65}{R}(S) = \frac{R-m}{R}$$

$$m = R\left(1 - cos\frac{28.65}{R}S\right)$$

(15.41)

Where:

R = Radius of horizontal Curve

S = Stopping Sight Distance

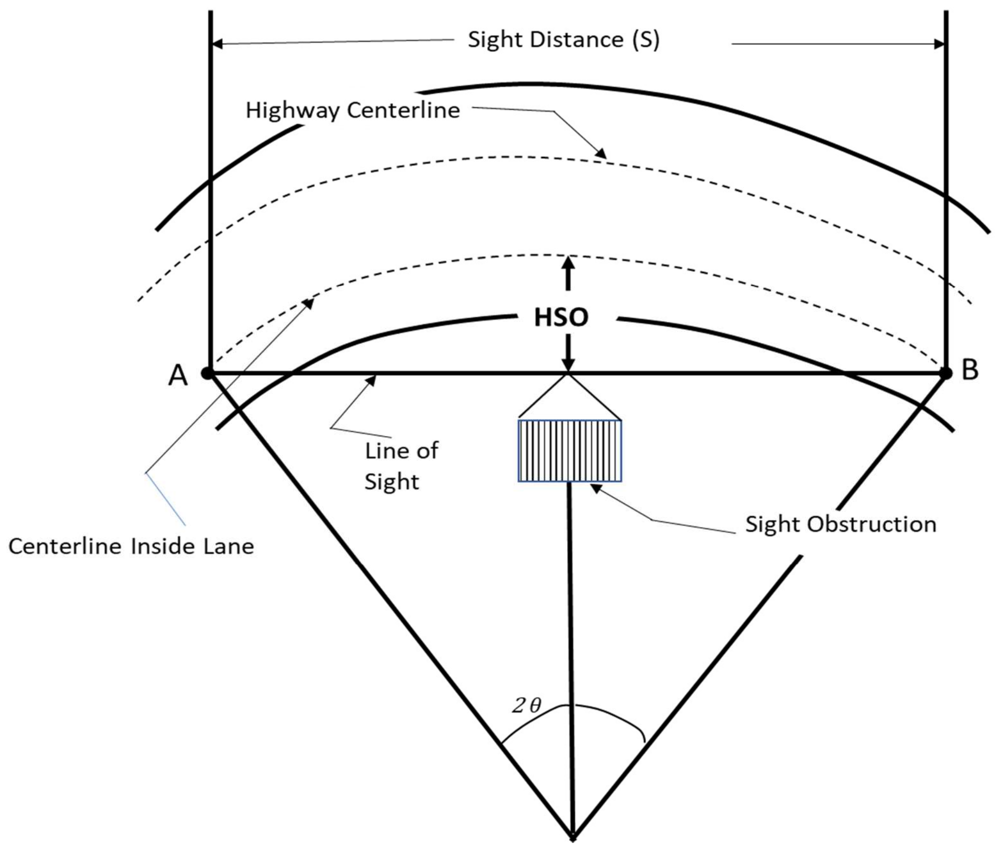
θ = One half central angle

From Figure 15.26(a), we can write:

$$\frac{R-m}{R} = \cos\theta$$

(15.42)

Figure 15.26 (a) is a systematic diagram of a horizontal curve with sight distance restrictions due to an object located within the curve line of sight.



Equating the cosines of θ from Eqs. 15.41 and 15.42 yields

$$cos\frac{28.65}{R}(S) = \frac{R-m}{R}$$

Where m = the Horizontal Sightline Offset, HSO (ft.). Solving for m produces the following relationship:

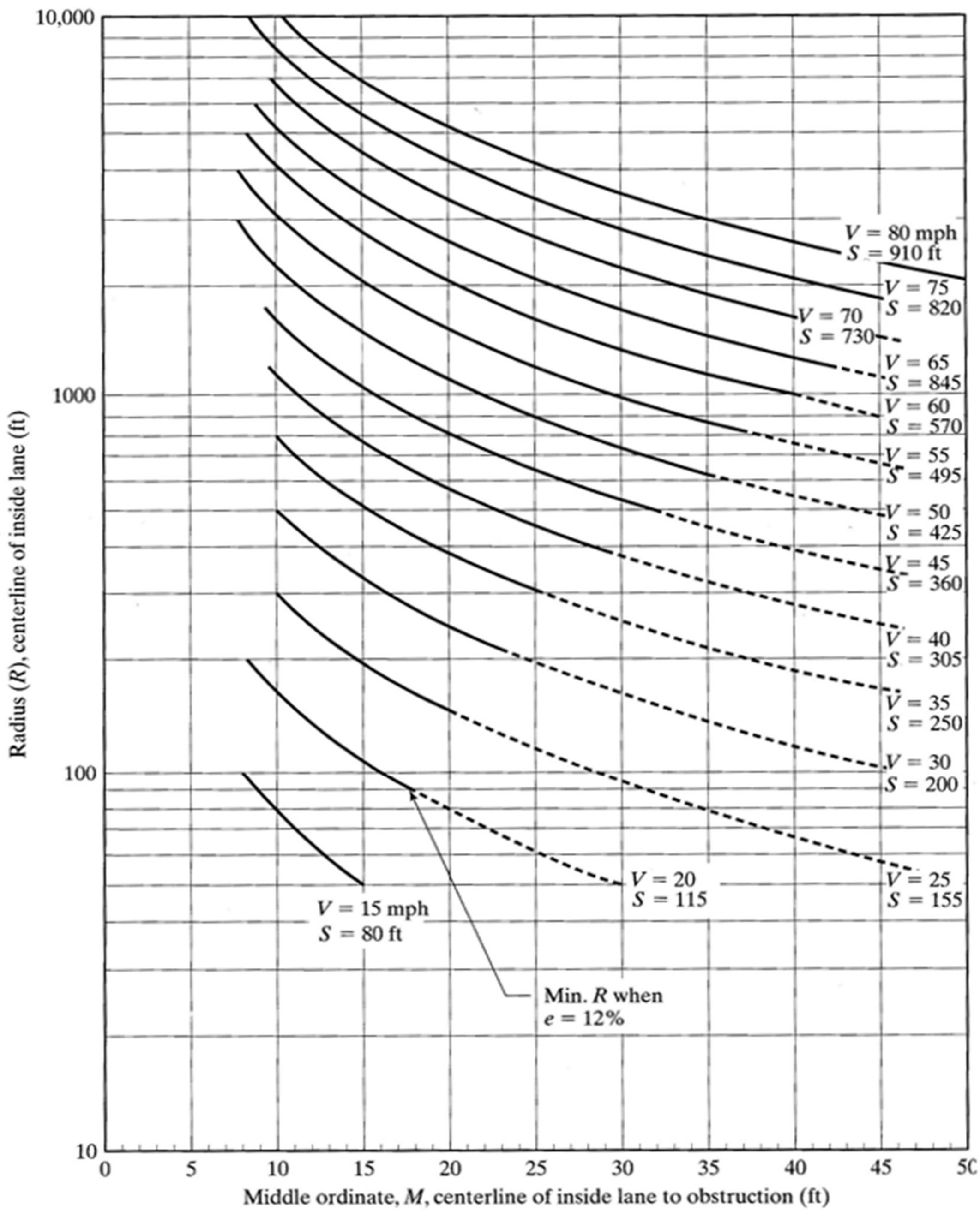
$$m = R\left(1 - cos\frac{28.65}{R}S\right)$$

(15.43)

Eq. 15.43 can be used to determine *m*, *R* or *S*, depending on which two of the three variables are known

Figure 15.26 (b) is a graphical representation of Eq. 15.43 and can be used to determine the value of the unknown variable.

Figure 15.26 (b) Horizontal Curve with sight-distance restrictions and Range of lower values for Stopping Sight Distances.



Example 15.8 Location of object near a Horizontal Curve.

A horizontal curve with a radius of 240m connects the tangents of a two-lane highway that has a posted speed limit of 56km/h. If the highway curve is not super elevated, e = 0, determine the horizontal sightline offset (HSO) that a large billboard can be placed from the centerline of the inside lane of the curve, without reducing the required SSD. Perception-reaction time is 2.5s, and f_b= 0.35.

Solution

Determine the required SSD.

$$SSD = 0.278\,ut + \,u^2/254(f \mp G)$$

$$(0.278 \times 56 \times 2.5) + \frac{(56)^2}{254(0.35)} = 74.20m$$

Determine m using Eq. 15.43

$$m = 240\left[1 - cos\left(\frac{28.65}{240}(74.20)\right)\right] = 240(1 - 0.988)m$$

$$= 2.86m$$

Check solution using Fig. 15.26(b)

For R=240m and V= 56km/h from fig.15.26 (b) m is estimated to be 2.85m

Horizontal Curve Widening

Along a highway horizontal curves often need to be made wider than the tangent sections of the road. The principle reasons for lane widening are large vehicles, whose bumpers have large overhangs and whose rear wheels track inside the paths taken by the lead wheels. This consideration is especially important on roadways where the lanes are less than 12 feet wide. The amount of widening needed depends on the following:

1 The design speed.
2 The design vehicle.
3 Horizontal curve radius.
4 Width of road.

If the traffic is principally passenger vehicles, then an extra 2 feet of lane width in the curve or turn, is satisfactory. If a large number of trucks or buses use the road, widening curves must account for the turning track of the design vehicle.

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

Where;

l= Length of wheel base of the design vehicle

n = Number of lanes

R = Radius of the Circular Curve

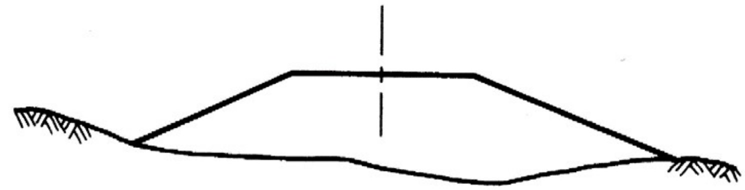
V = Design Speed

Highway Earthwork and Final Plans

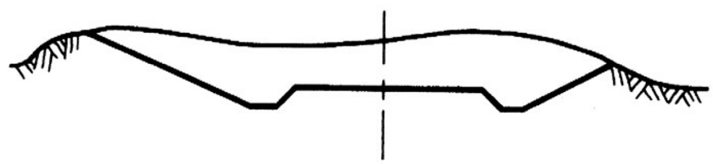
Highway Grades and Terrain.

- One factor that significantly influences the selection of a highway location is the terrain of the land, which in turn affects the laying of the grade line.
- The primary factor that the designer considers on laying the grade line is the amount of earthwork that will be necessary for the selected grade line. **The ultimate purpose is to minimize this amount.**
- One method to reduce the amount of earthwork is to set the grade line as closely as possible to the natural ground level.
- Another method to reduce the cost is by setting the grade line in such a way that there is a balance between the excavated volume and the volume of embankment.
- Another factor that should be considered in laying the grade line is the existence of fixed points, such as railway crossings, intersections with other highways, and in some cases existing bridges, which require that the grade be set to meet them.
- The height of the grade line is usually dictated by the expected floodwater level.
- Grades line should also be set in such a way that the minimum sight distance requirements are obtained.
- To determine the amount of earthwork involved for a given grade line, cross sections are taken at regular intervals along the grade line. The cross sections are usually spaced 15 to 30m apart, although this distance is sometimes increased for preliminary engineering.

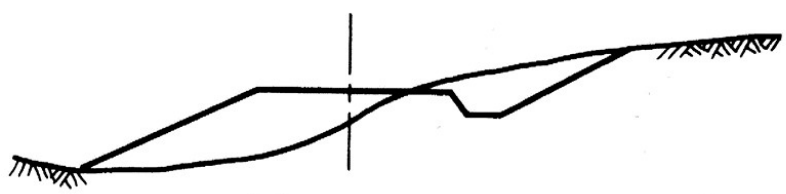
Figure 14.16 Types of Cross Sections



(a) Fill



(b) Cut



(c) Fill and Cut

These cross sections are obtained by plotting the natural ground levels and proposed grade profile of the highway along a line perpendicular to the grade line to indicate areas of excavation and areas of fill. Figure 14-16 shows three types of cross section. When the computation is done manually, the cross sections are plotted on standard cross-section paper, usually to a scale of I/50 or 1/100 for both the horizontal and vertical directions. The areas of cut and fill at each cross-section are then determined by the use of a planimeter or by any other suitable method.

Computing Earthwork Volumes.

A common method of determining the volume is that of average end areas. This procedure is based on the assumption that the volume between two consecutive cross sections is the average of their areas multiplied by the distance between them, computed as follows:

$$v = \left(\frac{A_1 + A_2}{2}\right) L$$

(14.7)

Where:

V=volume (m^3)

A_1 and A_2 = end areas (m^2)

L = Distance between cross sections (m)

Example:

The planimetered areas in m^2 of two hill-side cross-sections of a proposed highway are as follows:

Chainage	4200.0m	C82	F112
Chainage	4250.0m	C214	F78

C denotes cut, and F denotes fill. Calculate the quantities of earthwork.

Solution:

$$Distance\ L = 4250.0 - 4200 = 50.0m$$

$$Cut\ Volume = \left(\frac{82 + 214}{2}\right) \times 50 = 7400m^3$$

$$Fill\ Volume = \left(\frac{112 + 78}{2}\right) \times 50 = 4750m^3$$

Computing Earthwork Volumes.

- It is common practice in earthwork construction to move suitable material from cut sections to fill sections to reduce to a minimum the amount of material borrowed from borrow pits.
- When the materials excavated from cut sections are compacted at the fill sections they fill less volume than was originally occupied. This phenomenon is referred to as *Shrinkage* and should be accounted for when excavated material is to be reused as fill material.
- The amount of shrinkage depends on the type of material. Shrinkage of up to 50 percent have been observed for some soils. However, shrinkage factors used are generally between 1.10 and 1.25 for high fills and between 1.20 and 1.25 for low fills.

Volume before excavation = 1 m^2

Material	Volume immediately after excavation	Volume after compaction
Rock (large pieces)	1.5	1.4
Rock (Small pieces)	1.7	1.35
Chalk	1.8	1.4
Clay	1.2	0.90
Gravel	1.0	0.92

Example 14.4 Computing Fill and Cut Volumes using the Average End-Area Method.

A road section is 600 m long (20 Stations). The cut and fill volumes are to be computed between each station. The table 14.1 (Pg. 726) lists the station numbers (column 1) and lists the end area values (m^2) between each station that are in cut (column 2) and that are in fill (column 3). Material in a fill section will consolidate (known as shrinkage), and for this road section, is 10 percent. (For example, if 100 m^3 of net fill is required, the total amount of fill material that is supplied by a cut section is $100 + (0.10 \times 100) = 100 + 10 = 110\ m^3$)

Determine the net volume of cut and fill that is required between Station 0 and Station 1

Solution:

$$V_{cut} = \frac{30(A_{0c} + A_{1c})}{2} = \frac{30(3+2)}{2} = 75m^3$$

$$V_{fill} = \frac{30\left(A_{0f} + A_{1f}\right)}{2} = \frac{30(18+50)}{2} = 1020m^3$$

$$\text{Shrinkage} = 1020(0.10) = 102m^3$$

$$\text{Total fill volume} = 1020 + 102 = 1122m^3$$

The cut and fill volume between station 0+00 and 1+00 is shown in column 4 and 7.

Cut: 75 m^3 (column 4)

Fill: 1020 m^3 (column 5)

Shrinkage: 102 m^3 (column 6)

Total fill required: 1122 m^3 (column 7)

Table 14.1 Computation of Fill and Cut Volumes and Mass Diagram Ordinate.

End Area (m^2)			Volume (m^3)				Net Volume (4 to 7)		
1	2	3	4	5	6	7	8	9	10
Station	Cut	Fill	Total Cut	Fill	Shrinkage 10 percent	Total Fill (5+6)	Fill (-)	Cut (+)	Mass Diagram Ordinate
0	3	18	-	-	-	-		-	0
1	2	50	75	1020	102	1122	1047	-	-1047
2	2	97	60	2205	221	2426	2366	-	-3413
3	4	130	90	3405	341	3746	3656	-	-7069
4	8	51	180	2715	272	2987	2807	-	-9876
5	40	45	720	1485	149	1634	914	-	-10790
6	45	20	1275	975	98	1073	-	202	-10588
7	80	5	1875	375	38	413	-	1462	-9126
8	122	2	3030	105	11	116	-	2914	-6212
9	130	0	3780	30	3	33	-	3747	-2465
10	140	0	4050	0	0	0	-	4050	1585
11	100	3	3600	45	5	50	-	3550	5135
12	80	30	2700	495	50	545	-	2155	7290
13	75	20	2325	750	75	825	-	1500	8790
14	50	50	1875	1050	105	1155	-	720	9510
15	20	80	1050	1950	195	2145	1095	-	8415
16	10	100	450	2700	270	2970	2520	-	5895
17	0	120	150	3300	330	3630	3480	-	2415
18	3	120	45	3600	360	3960	3915	-	-1500
19	40	50	645	2550	255	2805	2160	-	-3660
20	30	30	1050	1200	120	1320	270	-	-3930

Net volume between stations 0-1 = total cut – total fill = 75 – 1122
= -1047m^3(column 8)

Note: Net fill volumes are negative (-) (column 8) and net cut volumes are (+) (column 9)

Similar calculations are performed between all other stations, from station 1 + 00 to 20 + 00, to obtain the remaining cut or fill values shown in columns 2 through 9.

Computing Ordinates of the Mass Diagram.

- The mass diagram is a series of connected lines that depicts the net accumulation of cut or fill between any two stations. The ordinate of the mass diagram is the net accumulation in cubic meter (m^3) from an arbitrary starting point.
- Thus, the difference in ordinates between any two stations represents the net accumulation of cut and fill between these stations. If the first station of the roadway is considered to be the starting point, then the net accumulation at this station is zero.

Example 14.5 Computing Mass Diagram Ordinates

Use the data obtained in example 14.4 to determine the net accumulation of cut or fill beginning with station 0+00. Plot the results.

Solution:

Columns 8 and 9 show the net cut and fill between each station. To compute the mass diagram ordinate between station X and X+1, add the net accumulation from station X (the first station) to the net cut or fill volume (columns 8 or 9) between stations X and X +1. Enter this value in column 10.

Station 0+00 mass diagram ordinate = 0

Station 1+00 mass diagram ordinate = 0 – 1047 = -1047 m^3

Station 2+00 mass diagram ordinate = -1047 – 2366 = -3413 m^3

Station 3+00 mass diagram ordinate = -3413 – 3656 = -7069 m^3

Station 4+00 mass diagram ordinate = -7069 – 2807 = -9876 m^3

Station 5+00 mass diagram ordinate = -9876 – 914 = -10790 m^3

Station 6+00 mass diagram ordinate = 10790 + 202 = -10588 m^3

Station 7+00 mass diagram ordinate = -10588 – 1462 = -9126 m^3

Continue the calculation process for the remaining 13 stations to obtain the values shown in column 10 of table 14.1. A plot of the results is shown in fig. 14.17.

Interpretation of the Mass Diagram.

1. When the mass diagram slopes downwards (negative, the preceding section is in fill, and when the slope is upward (positive), the preceding section is in cut.
2. The difference in mass diagram ordinates between any two stations represents the net accumulation between the two stations (cut or fill). For example, the net accumulation between station 6+00 and 12+00 is 10588 + 7290 = 17878 m^3.
3. A horizontal line on the mass diagram defines the locations where the net accumulation between these two points is zero. These are referred to as "Balance Points" because there is a balance in cut and fill volumes between these points. In figure 14.17, the "X" axis represents a balance between points A' and D' and a balance between points D' and E'. Beyond point E', the mass diagram indicates a fill condition for which there is no compensating cut. The maximum value is the ordinate at station 20 + 00 of -3930 m^3(fill). For this section, imported material (called borrow) will have to be purchased and transported from an off-site location.
4. Other horizontal lines can be drawn connecting portions of the mass diagram. For example, lines J-K and S-T, which are each five stations long, depict a balance of cut and fill between stations at points J and K and S and T.
5. A negative value at the end of the curve indicates that borrow is required to complete the fill and a positive value at the end of the curve indicates the waste operation will be net result
6. Steep slope indicates high cut or fill. whereas zero slope indicates change from cut to fill or visa versa.
7. Zero value indicates a balance between cut and fill.
8. The highest or lowest points mean haul diagram represent the crossing points between the grade line (road level) and natural ground level.

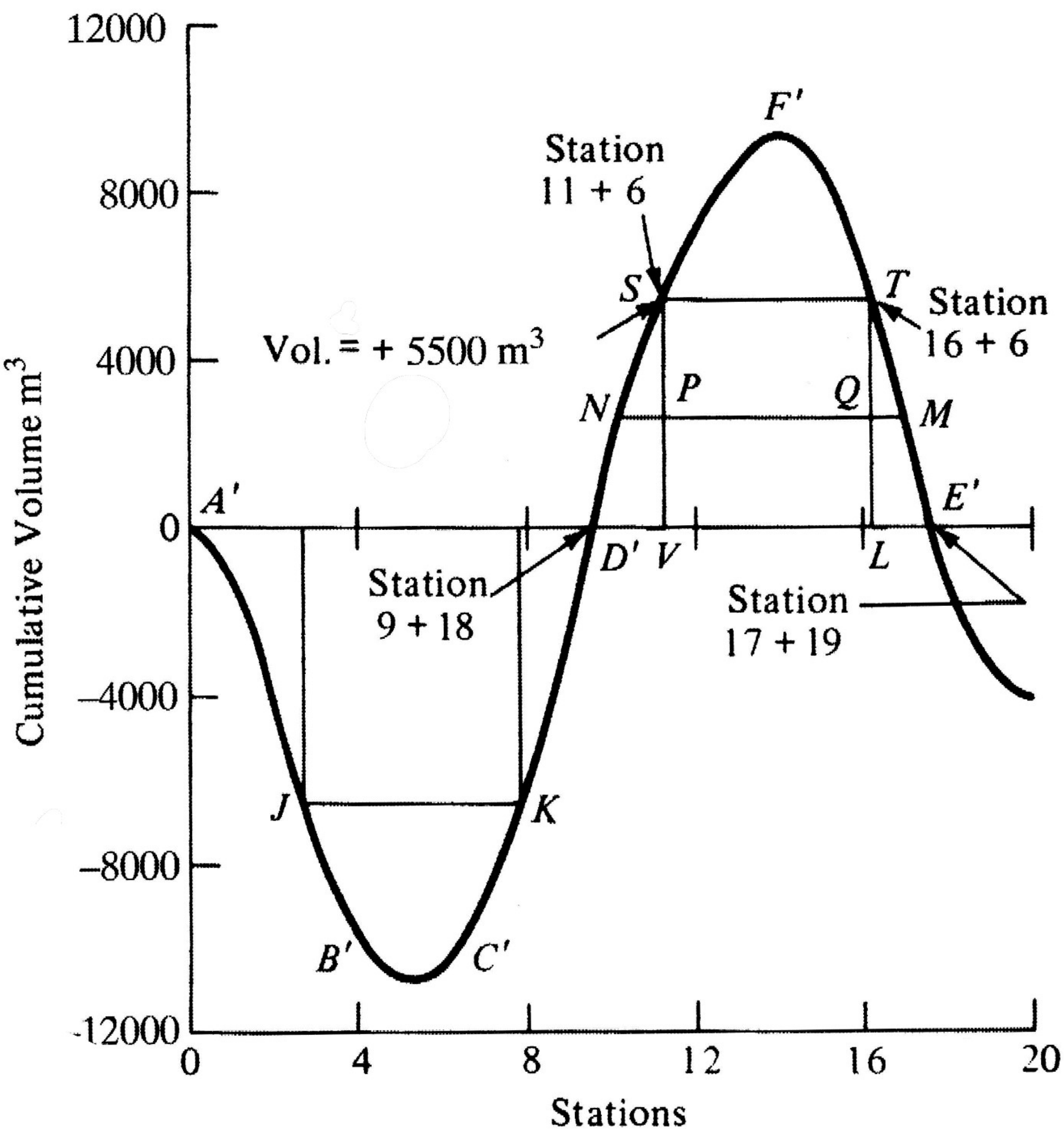


Figure 14.7 Mass Diagram for Computation Shown in Table 14.1

Example 14.6 Computing Balance Point Stations.

Compute the value of balance point stations for the mass diagram in figure 14.17 for the following situations:

(a) The X-axis

(b) The horizontal distance S-T, which measures 150 m.

Solution:

(a) Balance points are computed by interpolation using the even stations where the ordinates change from cut to fill (or vice versa).

Balance point D' occurs between Station 9+00 and 10+00 (since ordinate values are --2465 and +1585)

Assuming that the mass diagram ordinate changes linearly between stations, by similar triangles, we can write:

Station of the Balance Point

$$D' = (9 + 00) + \left[\frac{2465}{(2465 + 1585)}\right](30) = 9 + 18$$

Similarly,

Station of the Balance Point

$$E' = (17 + 00) + \left[\frac{2415}{(2415 + 1500)}\right](30) = 17 + 19$$

(b) To determine the balance point stations for line *ST*, it is necessary to draw the mass diagram to a larger scale than depicted in the textbook, and to read the station for one of the points directly from the diagram. Using this technique, Station 11 + 6 was measured for points *S* and from this value the station for point *T* is computed as:

$$(11 + 6) + (5 + 00) = Station\ 16 + 6$$

Mass Haul Diagram

To construct the mass haul diagram the following definitions are necessary:

1. **Haul:** The distance over which the material is moved; also represents the volume-distance of material moved.
2. **Free Haul Distance (F.H.D):** The distance within which a fixed price is paid for excavating, hauling and dumping of material regardless of its length (usually 150 – 350) m.
3. **Overhaul Distance (O.H.D):** The haulage distance beyond free haul distance for which an extra charge is required for each cubic meter.
4. **Max. Overhaul distance:** When the haul distance is great (larger than max. overhaul distance), it may be more economical to waste (cut) good excavation material and import (barrow) fill from a more convenient source rather than paying for overhauling.
5. **Limit of Economic Haul Distance (L.E.H.D):** The maximum overhaul distance plus free-haul distance beyond which it is more economical to waste and borrow rather than to pay for overhauling.

   L.E.H.D = F.H.D + Max. O.H.D

Parking Studies

Overview:

Parking is one of the major problems that is created by the increasing road traffic. The availability of less space in urban areas has increased the demand for parking space especially in areas like the central business district.

The Parking facilities can be grouped in the following two types.

1 kerb or on -street parking

2 off-street parking.

On street parking:

On street parking means the vehicles are parked on the sides of the street itself. This will be usually controlled by government agencies themselves. Common types of on-street parking are as listed below. This classification is based on the angle in which the vehicles are parked with respect to the road alignment.

The disadvantages of on- street parking are.

- The safety of various classes of road users is adversely affected (increase road accidents particularly pedestrian)
- Imped traffic flow and reduce their speed.
- Decrease the road capacity

The on- street parking can be permitted in a specified manner only. It is either.

a) parallel parking

b) angular parking

1. **Parallel parking**. The vehicles are parked along the length of the road. Here there is no backward movement involved while parking or un-parking the vehicle. Hence it is the safest form of parking from the accident prospective. However, it consumes the maximum Kerb length and therefore only a minimum number of vehicles can be parked for a given Kerb length. This method of parking produces least obstruction to the on-going traffic on the road since the least road width is used. Parallel parking of cars is

shown in Figure 15.33. The number of vehicles could park for a given Kerb length (in ft) is,

$$N = \frac{L}{22}$$

2. **30° Parking:** In thirty-degree parking, the vehicles are parked at 30° with respect to the road alignment. In this case, more vehicles can be parked compared to parallel parking. Also, there is better maneuverability. Delay caused to the traffic is also minimum in this type of parking. An example is shown in figure 15.33.

   For *N* vehicles,

$$N = \frac{L - 2.8}{17}$$

3. **45° Parking**: As the angle of parking increases, a higher number of vehicles can be parked. Hence compared to parallel parking and thirty-degree parking, a higher number of vehicles can be accommodated on this type of parking. figure. 15.33.

$$N = \frac{L - 6.7}{12}$$

4. **60° Parking:** The vehicles are parked at 60° to the direction of the road. A higher number of vehicles can be accommodated in this type of parking. From figure. 15.33, for length (L in ft) number of vehicles could park is,

$$N = \frac{L - 6.6}{9.8}$$

5. **Right Angle Parking:** In right angled or 90° parking, the vehicles are parked perpendicular to the direction of the road. Although it consumes maximum width, Kerb length required is very little. In this type of parking, the vehicles need complex maneuvering and this may cause severe accidents. This arrangement causes obstruction to the road traffic particularly if the road width is less.

However, it can accommodate maximum number of vehicles for a given Kerb length. An example is shown in figure 15.33.

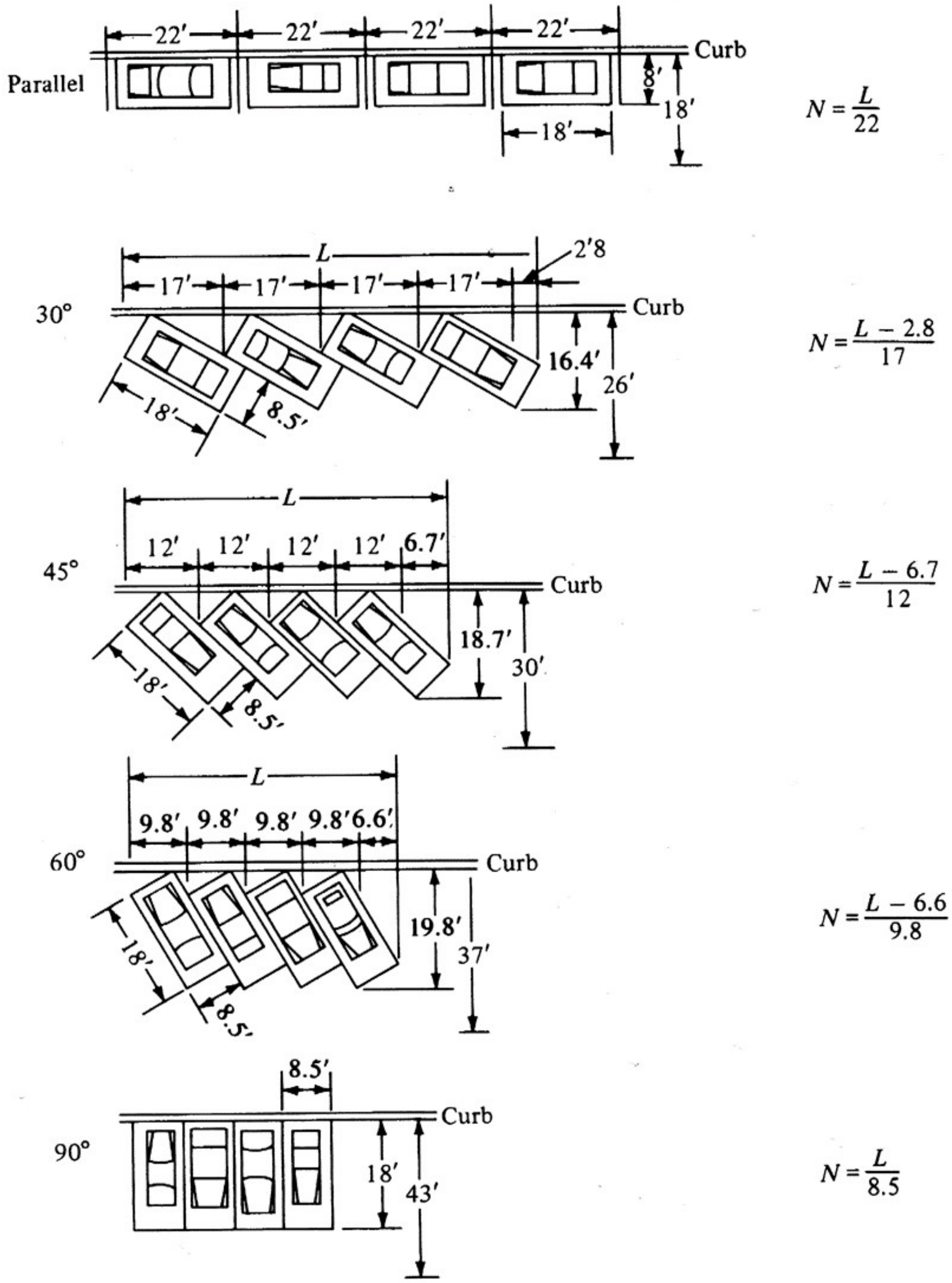
$$N = \frac{L}{8.5}$$

<u>Angular parking is designed under the following conditions</u>

- Wide streets having good sight distance
- Local streets.
- One-way streets.
- Low traffic volume.

Design of On-Street Parking Facilities.

The number of parking bays that can be fitted along a given length of Kerb increases as the angle of inclination increases from parallel (0 degrees) to perpendicular (90 degrees)



$N = Number\ of\ Spaces$
$L = Kerb\ Length$

Figure 15.33 street space used for various parking configuration.

Design of Off-Street Parking Facilities – Surface Car Parks

The layouts shown indicate that parking spaces are efficiently used when the parking bays are inclined at 90 degrees to the direction of traffic flow.

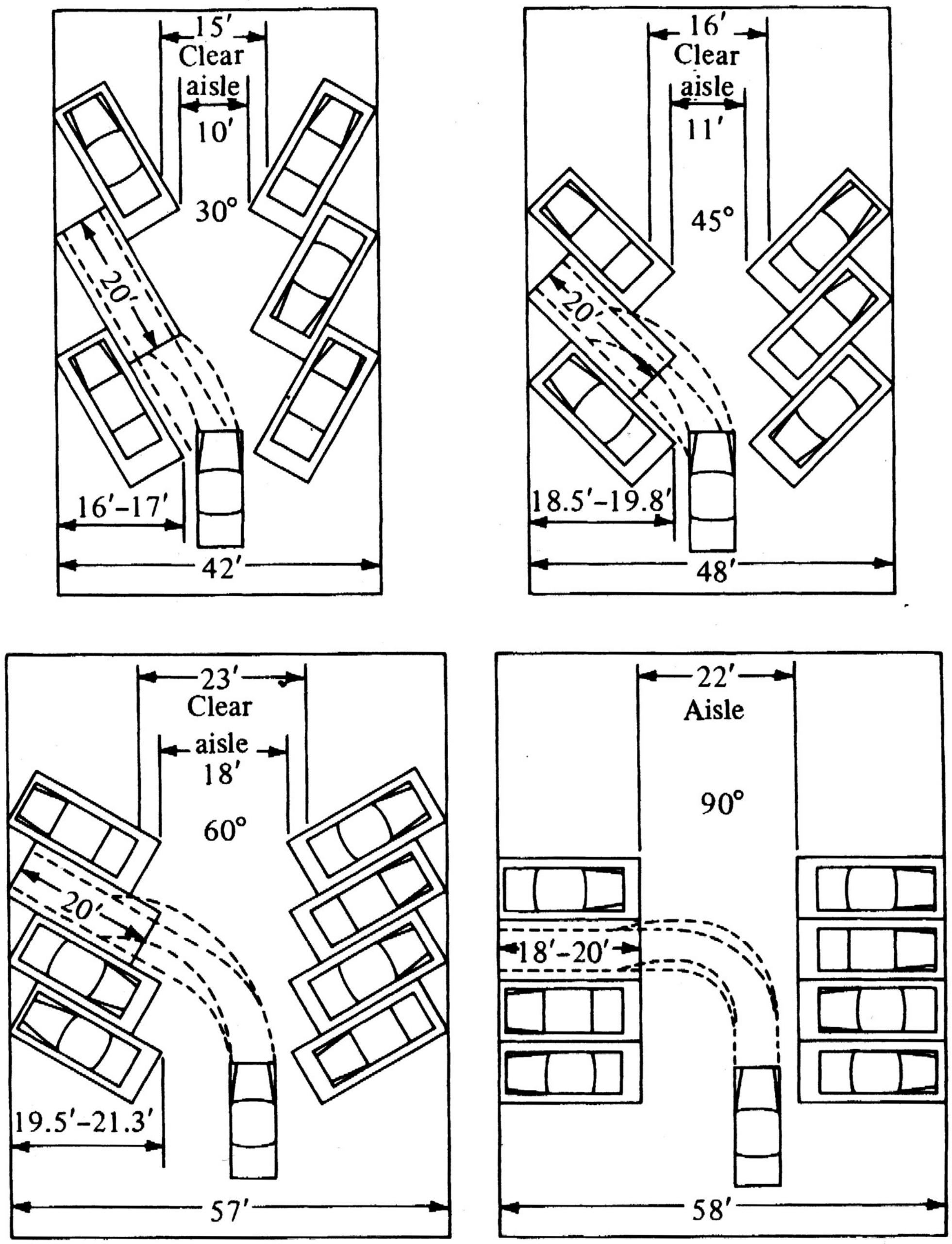


Figure 15.34 Parking Stall Layout

The use of the herringbone layout as illustrated here facilitates traffic circulation because it provides for one-way flow of traffic on each aisle

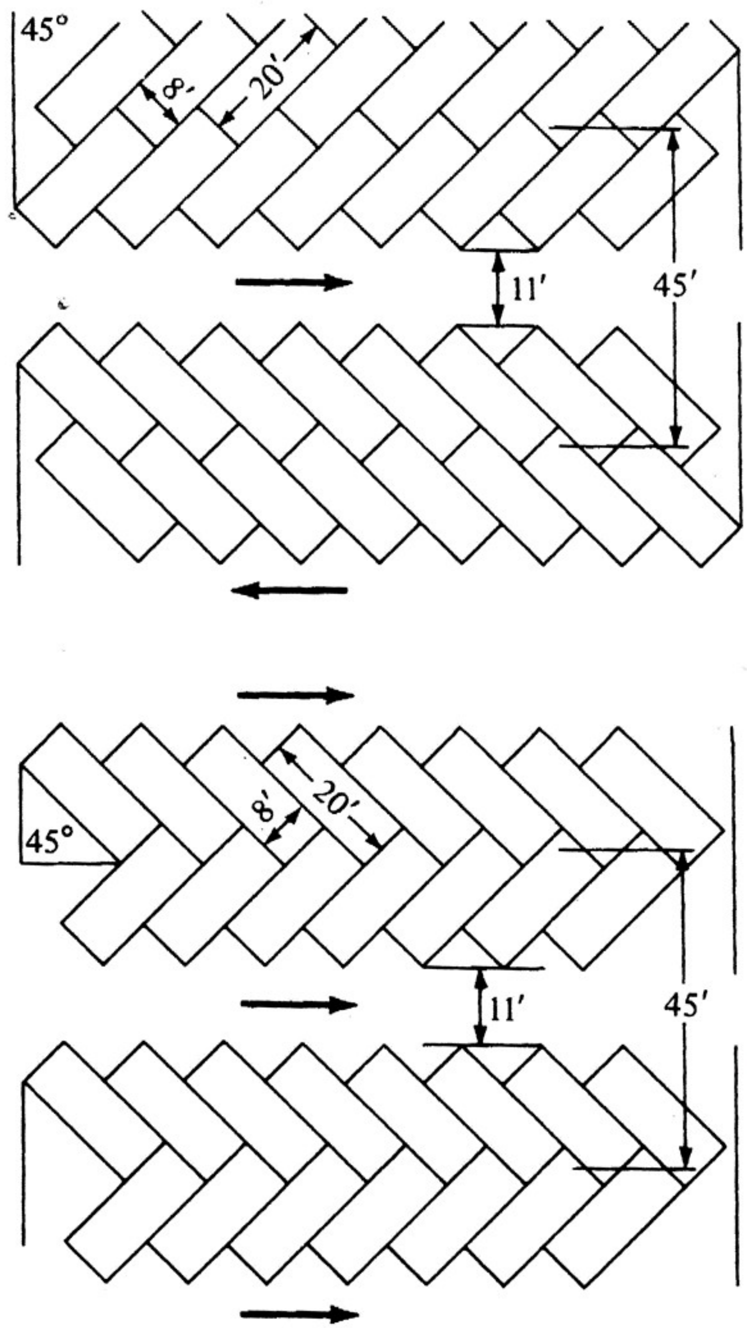


Figure 15.35 Herringbone layout of parking Stalls in an on-surface lot.

Design of Off-Street Parking Facilities –Garages

- Parking garages consist of several platforms, supported by columns, which are placed in such a way as to facilitate an efficient arrangement of parking bays and aisles.

- Access ramps connect each level with the one above. The gradient of these ramps is usually not greater than 1:10 on straight ramps and 1:12 on the centerline of curved ramps.

- The radius of curved ramps measured to the end of the outer curve should not be less than 21 m, and the maximum superelevation should be 0.15 m/m. The lane width should not be less than 4.8 m for curved ramps and 2.7 m for straight ramps.

- Ramps can be one-way or two-way, with one-way ramps preferred. When two-way ramps are used, the lanes must be clearly marked and where possible physically divided at curves and turning points to avoid head-on-collisions, as drivers may cut corners or swing wide at bends.

Off-Street Parking Facilities.

These facilities may be privately or publicly owned; they include surface lots and garages. Self-parking garages require that drivers park their own vehicles; attendant -parking garages maintain personnel to park the vehicles.

Definition of Parking Terms

Before discussing the different methods for conducting a parking study, it is necessary to define some terms commonly used in parking studies including space-hour, parking volume, parking accumulation, parking load,

Parking duration, and parking turnover.

1) A **Space-hour** is a unit of parking that defines the use of a single parking space for the period of 1 hour.
2) **Parking Volume** is the total number of vehicles that park in a study area during a specific length of time, usually a day.
3) **Parking Accumulation** is the number of parked vehicles in a study area at any specified time. These data can be plotted as a curve of parking accumulation against time, which shows the variation of the parking accumulation during the day.
4) The **Parking Load** is the area under the accumulation curve between two specific times. It is usually given as the number of space-hours used during the specified period of time.
5) **Parking Duration** is the length of time a vehicle is parked at a parking bay. When the parking duration is given as an average, it gives an indication of how frequently a parking space becomes available.
6) **Parking Turnover** is the rate of use of a parking space. It is obtained by dividing the parking volume for a specified period by the number of parking spaces.

Example

In a parking garage which has a capacity of 300 parking spaces and open from 7am – 7pm. There are 480 vehicles parked daily. 30% of the parked vehicles parked 8hrs, 50% parked for 6hr, and 20% parked for 2hrs. Determine whether there is a need to provide for additional parking spaces for the garage.

Solution:

$$480 \times 30\% = 144\ vehicle\ parking\ for\ 8\ hrs$$
$$Space - hrs = 144 \times 8 = 1152\ space - hrs$$
$$480 \times 50\% = 240\ vehcle\ parking\ for\ 6\ hrs$$
$$Space - hrs = 240 \times 6 = 1440\ space\ hrs$$
$$480 \times 20\% = 96\ vehicle\ parking\ for\ 2\ hrs.$$
$$Space - hrs = 96 \times 2 = 192\ space\ hrs$$
$$Total\ Space - hrs = 1552 + 1440 + 192 = 2784\ space\ hrs.$$
$$The\ garage\ capacity = 300$$
$$Space - hr\ avaliable = 300 \times 12 = 3600\ Space - hrs.$$
$$3600 > 2784$$

Therefore, there is no need for any additional spaces

Example 4.7 Space Requirements for a Parking Garage.

The owner of a parking garage located at a CBD has observed that 20% of those wishing to park are turned back every day during the opening hours of 8 am to 6 pm because of lack of parking spaces. An analysis of data collected at the garage indicates that 60% of those who park are commuters, with an average parking duration of 9 hours and the remaining are shoppers, whose average parking duration is 2 hours. If 20% of those who cannot park are commuters and the rest are shoppers and a total of 200 vehicles currently park daily in the garage, determine the number of additional spaces required to meet the excess demand. Assume parking efficiency is 0.90.

Solution:

- Calculate the space-hours of demand using Eq. 4.12

$$D = \sum_{i=1}^{N} (n_i t_i)$$

Commuters now being served = $0.6 \times 200 \times 9$ = 1080 space-hr
Shoppers now being served = $0.4 \times 200 \times 2$ = 160 space-hr
Total number of vehicles turned away $= \frac{200}{0.8} - 200 = 50$
Commuters not being served = $0.2 \times 50 \times 9$ = 90 space-hr
Shoppers not being served = $0.8 \times 50 \times 2$ = 80 space-hr
Total space-hours of demand = (1080 + 160 + 90 +80) = 1410
Total space-hours served = 1080 + 160 = 1240
Number of space-hours required = 1410 – 1240 = 170

- Determine the number of parking spaces required from Eq. 4.13.

$$S = f\sum_{i=1}^{N} t_i = 170\ space - hr$$

- Use the length of time each space can be legally parked on (8 am through 6 pm = 10 hr) to determine the number of additional spaces.

$$0.9\ \times 10\ \times N = 170$$

$$N = 18.89$$

At least 19 additional spaces will be required since a fraction of a space cannot be used.

Intersection Design

An intersection is an area, shared by two or more roads, whose main function is to provide for the change of route directions.
Intersections vary in complexity from:

- Simple intersection: Has only two roads crossing at a right angle.

- Complex Intersection: Three or more roads cross within the same area.

- Drivers therefore have to make a decision at an intersection concerning which of the alternative routes they wish to take.

- Intersections tend to have a high potential for crashes.

The overall traffic flow on any highway depends to a great extent on the performance of the intersections. since intersections usually operate at a lower capacity than through sections of the road.

Intersections are classified into three general categories:

- Grade-separated without ramps
- Grade-separated with ramps (Commonly known as interchanges)
- at-grade intersection.

Basic forms of at grade Intersection.

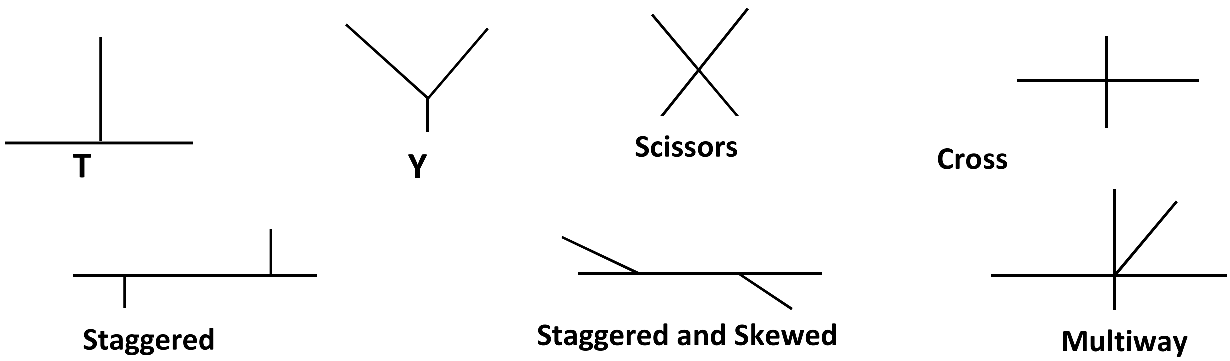
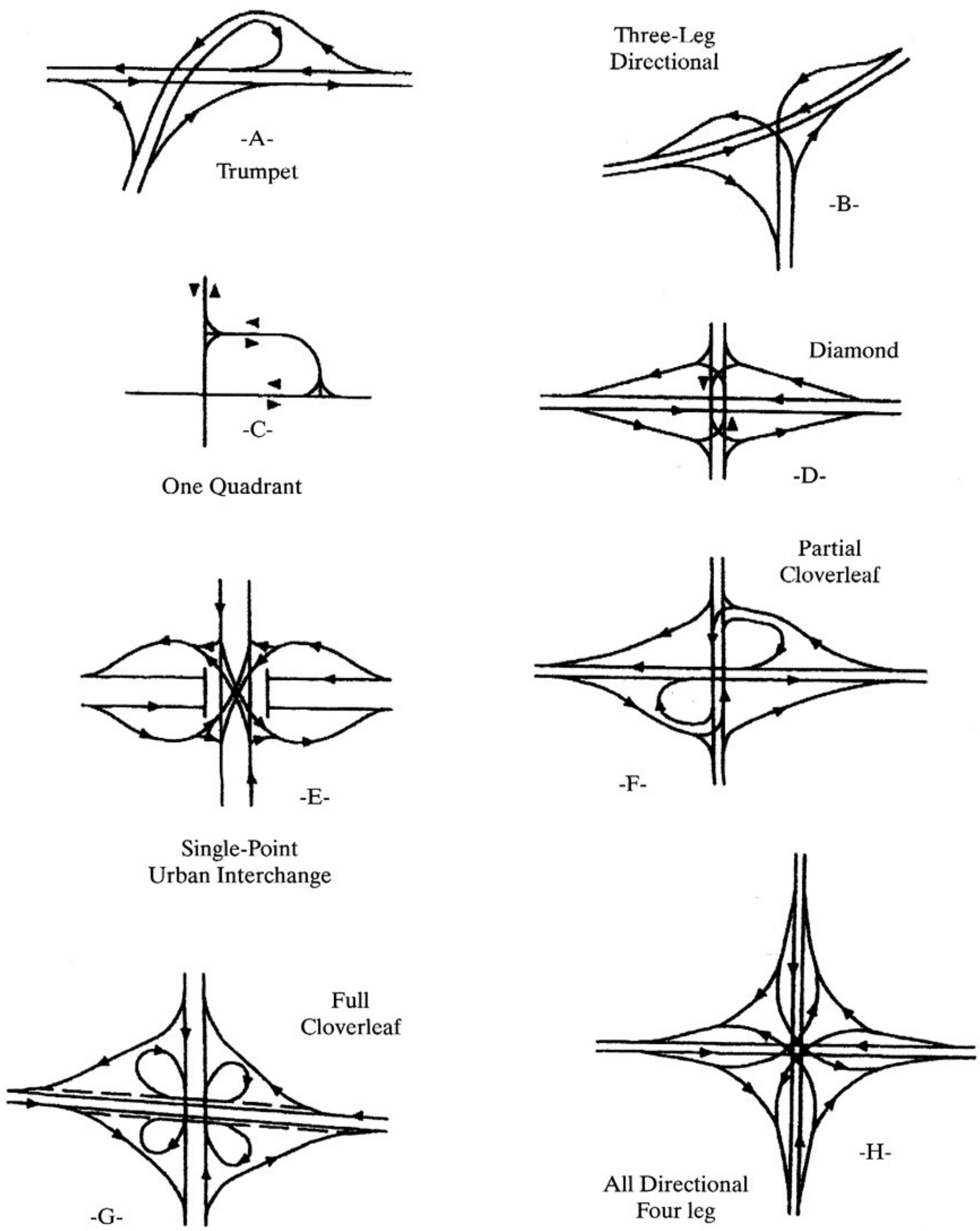
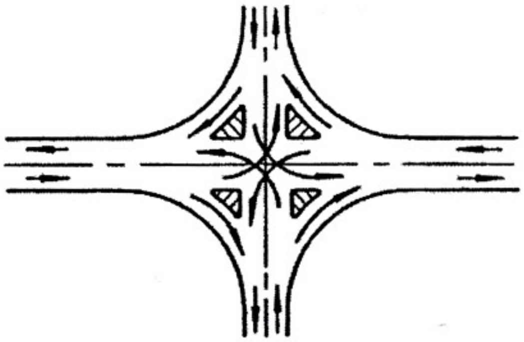


Figure 7.1 Shows different types of grade separated intersections.

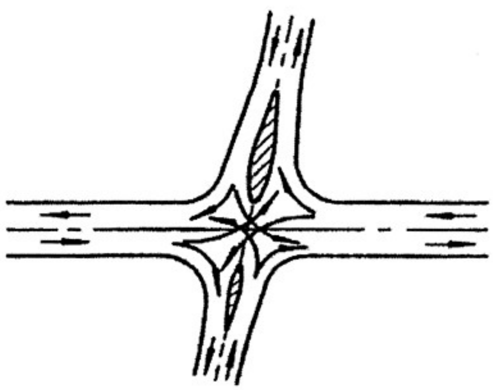
Figure 7.1 Examples of Grade Separated Interchanges



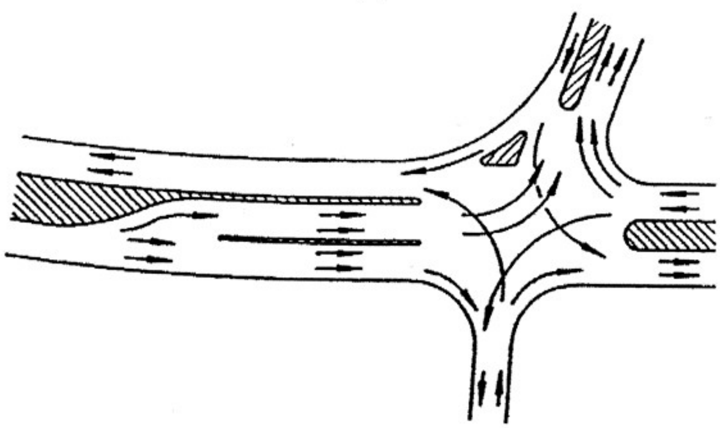
Figures 7.2 show different types of at-grade intersections.



(a)



(b)



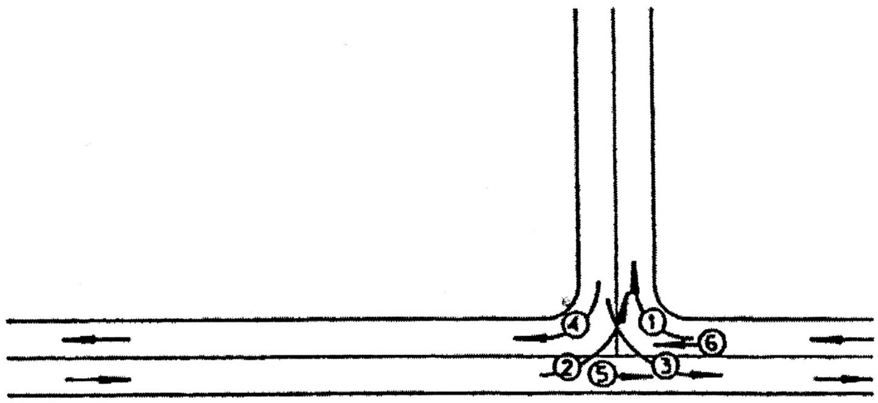
Types of At-Grade Intersections.

- The basic types of at-grade intersections are T or three-leg intersections which consist of three approaches;
- Four-leg or cross intersections, which consist of four approaches;
- multi-leg intersections, which consist of five or more approaches.

7.1.1 T Intersections

Figure 7.4 on page 270 shows different types of T intersections.
Simplest shown in figure 7.4a channelized one with divisional islands and turning roadways shown in Figure 7.4d.
Channelization involves the provision of facilities such as pavement markings and traffic islands to regulate and direct conflicting traffic streams into specific paths.

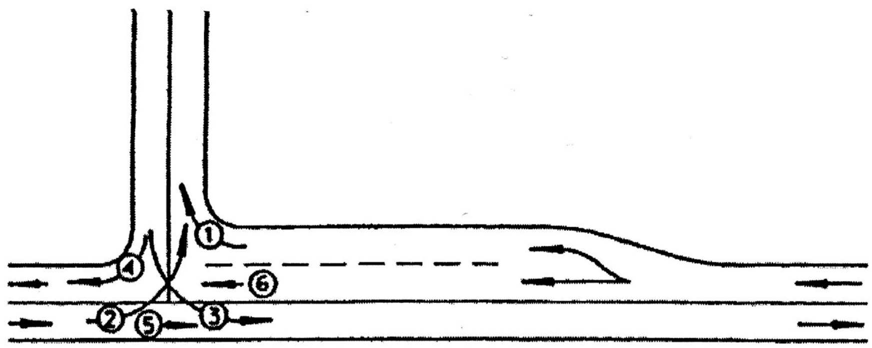
Figure 7.4 Examples of T Intersections



(a) Plain 'T' Intersection

The intersections shown in figure 7.4a is

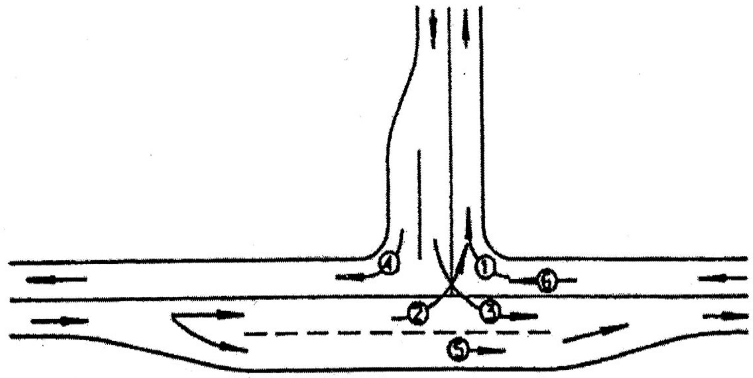
- suitable for minor or local roads.
- it is used when minor roads intersect important highways with intersection angle less than 30 degrees from the normal.
- They are also suitable for use in rural two-lane highways that carry light traffic.



(b) 'T'Intersection (With Right Turn Lane)

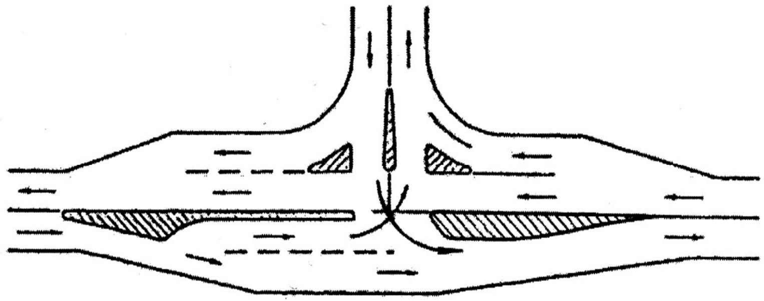
At locations with higher speeds and turning volumes, which increase the potential of rear-end collisions between through vehicles and turning vehicles. usually an additional area of surfacing or flaring is provided, as shown in 7.4b.

The flare is provided to separated right-turning vehicles from through vehicles approaching from the east.



(c) 'T' Intersection (With Right-Hand Passing Lane)

In cases where left-turn volume from a through road onto a minor road is sufficiently high but does not require a separate left-turn lane, an auxiliary lane may be provided, as shown in figure 7.4c.
This provide the space needed for through vehicles to maneuver around left-turning vehicles which have to slow down before making their turns.



(d) 'T' Intersection (With Divisional Island and Turning Roadways)

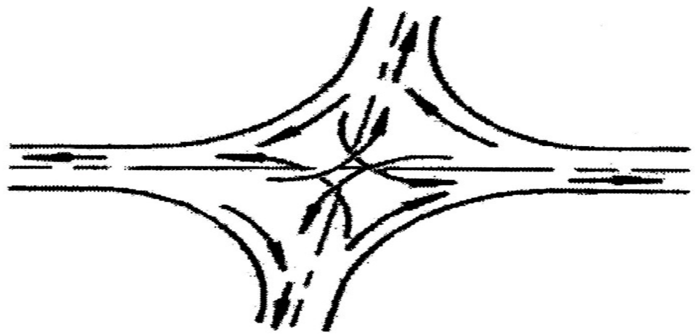
Figure 7.4d shows a channelized T intersection in which the two-lane through road has been converted into a divided highway through the intersection.

The channelized T intersection also provide both a left-turn storage lane for left turning vehicles from through road to the minor road and a right turn lane on the east approach. This type of intersection is suitable for location where volumes are high such as left -turn volume from through road and high right-turn onto the minor road

Intersection of this type probably will be signalized.

7.1.2 Four-leg Intersections

Figure 7.5 shows varying levels of channelization at a four-leg intersection.

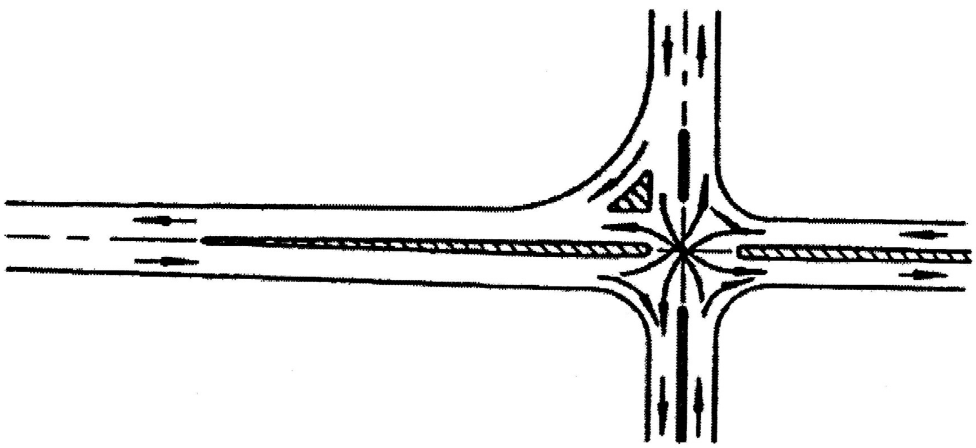


(a)

Un-channelized intersection shown in Figure 7.5a on page 272 is used mainly at

- locations where minor or local roads cross.
- the turning volumes are usually low
- low percentage of heavy vehicles
- low speed.
- It also can be used where a minor road crosses a major highway.

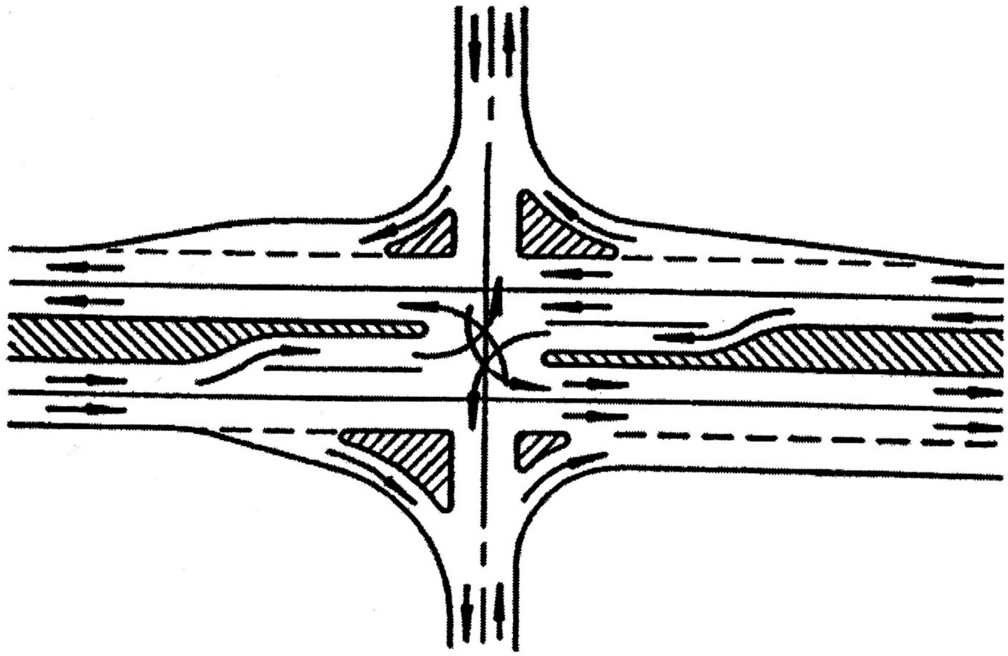
- When right-turning movements are frequent, right-turning roadways, such as those Figure 7.5b, can be provided.
- also, common where pedestrians are present.



(c)

The layout shown in figure 7.5c is suitable for:

- A two-lane highway that is not a minor crossroad
- and that carries moderate volumes at high speeds
- the flow operates near capacity.



(d)

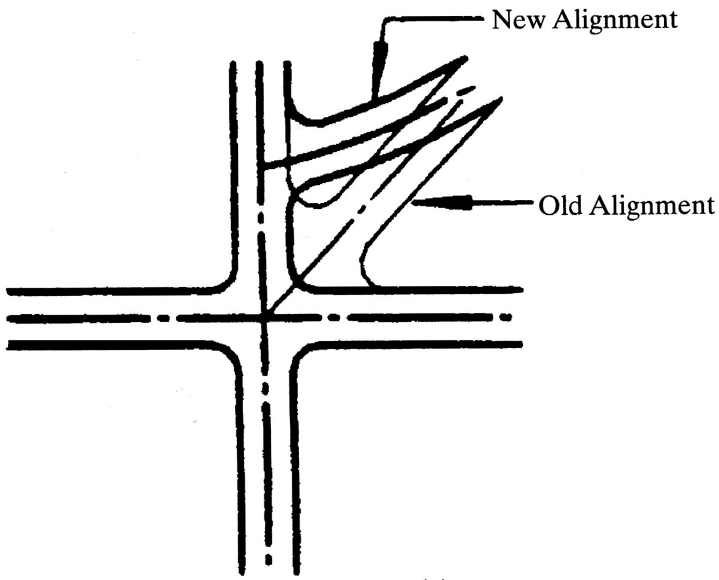
Figure 7.5 Examples of Four-Leg Intersections

Figure 7.5d shows a suitable design for

- four-lane approaches
- Carrying high through volumes and high left and right turning volumes
- This type of intersection is usually signalized.

7.1.3 Multi-Leg Intersections

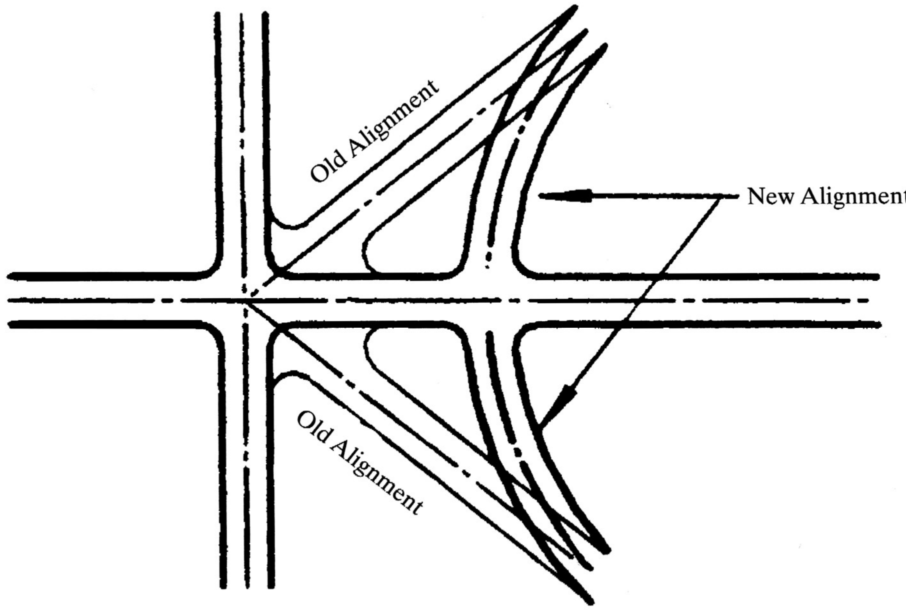
Multi-Leg Intersections have five or more approaches.



(a)

Whenever possible, this type of intersection should be avoided.
In order to Remove some: of the conflicting movements and increase safety and operation, one or more of the legs are realigned.
Figure 7.6a, the diagonal leg of the intersection is realigned.
This results in the formation of an additional T Intersection.
Two important factors to consider:

1) The diagonal road should be realigned to the minor road.
2) The distance between the intersections should be such that they can operate independently.



(b)

Figure 7.6 Examples of Multi-Leg Intersections

but with the multileg intersection now converted to a four-leg intersection.

- Realignment of a six -leg intersection Figure 7.6b, forming two four-leg intersections.
- Realignment to be made to the minor road forming two additional T intersections and resulting in a total of three intersections.
- The distances between these intersections should be great enough to allow for the independent operation of each intersection.

7.1.4 Traffic Circles

A traffic circle is a circular intersection that provides a circular traffic pattern with significant reduction in the crossing conflict points.

Types of traffic circles :

An informational guide, describes three types of traffic circles:

1. Rotaries.
2. Neighborhood traffic circles.
3. Roundabouts.

1) ***Rotaries*** have large diameters that are usually greater than 90 m, thereby allowing speeds exceeding 48 km/h, with a minimum horizontal deflection of the path of the through traffic.

2) ***Neighborhood traffic circles*** have diameters that are much smaller than rotaries and therefore allow much lower speeds.

Consequently, they are used mainly at the intersections of local streets, Traffic calming and/or aesthetic device.

Pedestrian may or may not be allowed

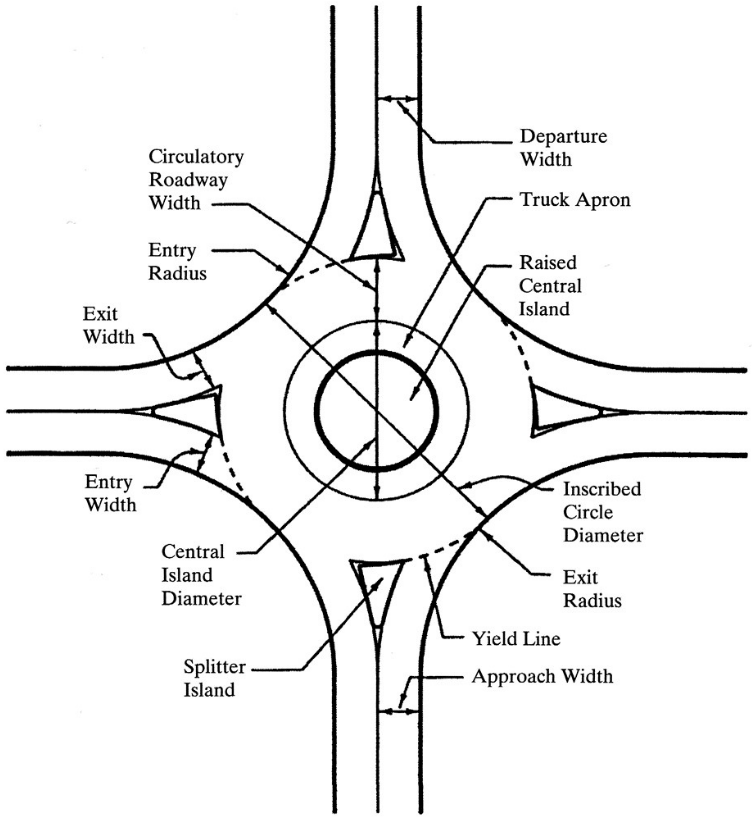
Parking may be allowed

They consist of pavement markings and do not usually employ raised islands.

3) **Roundabouts** have specific defining characteristics that separate them from other circular intersections. These include:

- Yield control at each approach
- Separation of conflicting traffic movements by pavement markings or raised islands.
- Geometric characteristics of the central island that typically allow travel speeds of less than 48 km/h
- Parking not usually allowed within the circulating roadway.

Figure 7.7a shows the geometric elements of a single-lane modern roundabout.



(a) Geometric Elements of a Single-Lane Modern Roundabout

Roundabouts can be further characterized into six classes based on the size and environment in which they are located.

1 Mini roundabouts
2 Urban compact roundabouts
3 Urban single-lane roundabouts
4 Urban double-lane roundabouts
5 Rural single-lane roundabouts
6 Rural double-lane roundabouts

Design Element	*Mini-Roundabout*	*Urban Compact*	*Urban Single-Lane*	*Urban Double-Lane*	*Rural Single-Lane*	*Rural Double-Lane*
Recommended maximum entry design speed	25 km/h (15 mi/h)	25 km/h (15 mi/h)	35 km/h (20 mi/h)	40 km/h (25 mi/h)	40 km/h (25 mi/h)	50 km/h (30 mi/h)
Maximum number of entering lanes per approach	1	1	1	2	1	2
Typical inscribed circle diameter[1]	13 to 25 m (45 ft to 80 ft)	25 to 30 m (80 to 100 ft)	30 to 40 m (100 to 130 ft)	45 to 55 m (150 to 180 ft)	35 to 40 m (115 to 130 ft)	55 to 60 m (180 to 200 ft)
Splitter island treatment	Raised if possible, crosswalk cut if raised	Raised, with crosswalk cut	Raised, with crosswalk cut	Raised, with crosswalk cut	Raised and extended, with crosswalk cut	Raised and extended, with crosswalk cut
Typical daily service volumes on four-leg roundabout (veh/day)	10,000	15,000	20,000	Refer to the source	20,000	Refer to the source

[1]Assumes 90° entries and no more than four legs.

SOURCE: *Roundabouts: An Informational Guide.* U.S. Department of Transportation, Federal Highway Administration, Publication No. FHWA-RD-00-067, Washington, D.C., 2000.

Advantages and Disadvantages of Roundabouts.

Advantages:

1 It provides a simple solution for road junctions where more than four roads meet and it proves to be advantageous when the number of intersecting roads is between four and seven.
2 It regulates traffic and there is a continuous flow of traffic through the roundabout.
3 Smooth and orderly flow of traffic with little delay.
4 It is cheaper to construct than a separated intersection.
5 Self- controlled traffic avoiding the need for traffic police or signals.

Disadvantages:

1 It requires a large area of flat land.
2 It is not possible to use it in congested areas.
3 Difficult for pedestrians.
4 It requires the instillation of complicated traffic signals.
5 Not suitable for high speed roads.

Conflict Points at Intersections.

Conflicts occur when traffic streams moving in different directions interfere with each other.

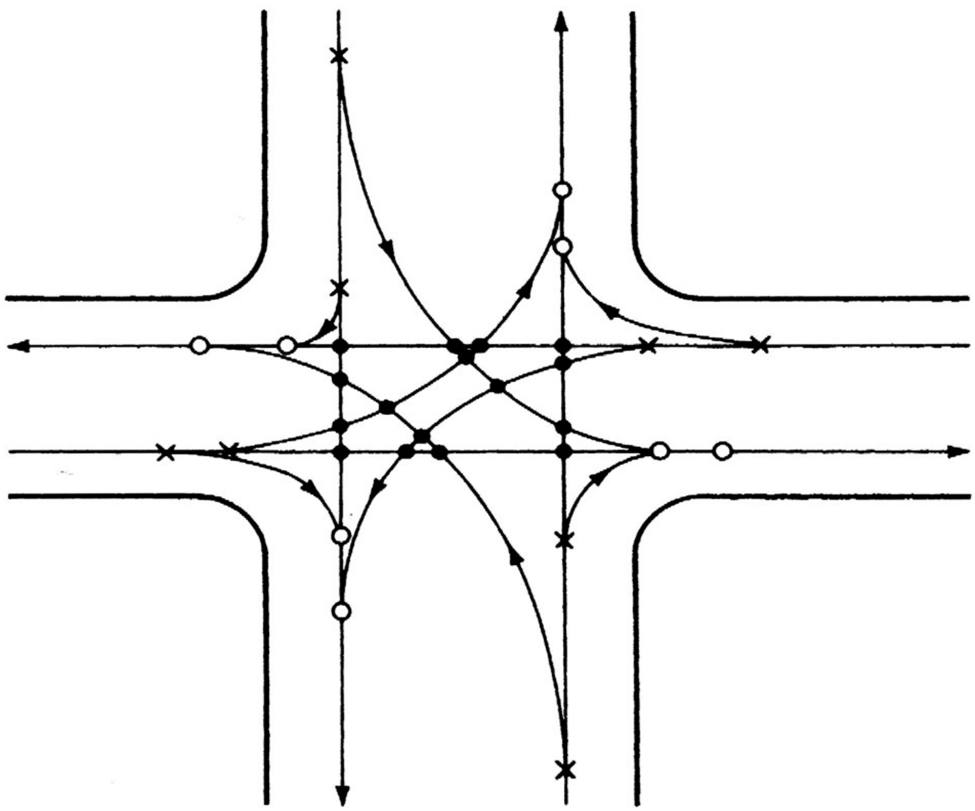
Three types of conflicts:

- Merging.
- Diverging.

* Crossing.

Figure 8.3 four-approach unsignalized intersection. There are 32 conflict points in this case.
The number of possible conflicts points at any intersection depends on:

* The number of approaches,
* The turning movements,
* The type of traffic control at the intersection.



o Merging conflict points = 8
× Diverging conflict points = 8
• Crossing conflict points = 16

Figure 8.3 Conflict Points at Four-approach Unsignalized Intersection

Crossing conflicts however, tend to have the most severe effect on traffic flow and should be reduced to a minimum whenever possible.

Design Principles for at-grade intresction.

- Minimize severity of potential conflicts.
- Provide for smooth flow of traffic.
- Consider both vehicles and pedestrians.
- Turning radius should not be less than the turning radius of the design vehicle or the radius required for the design speed.
- The design should ensure adequate sight distance.
- Intersection should not be located at or just beyond sharp horizontal and vertical curve.
- Angle of crossing should not be acute angle
- Give high priority to direction having high speed and volume.
- Determination of number of lanes (Provision of turning lanes)

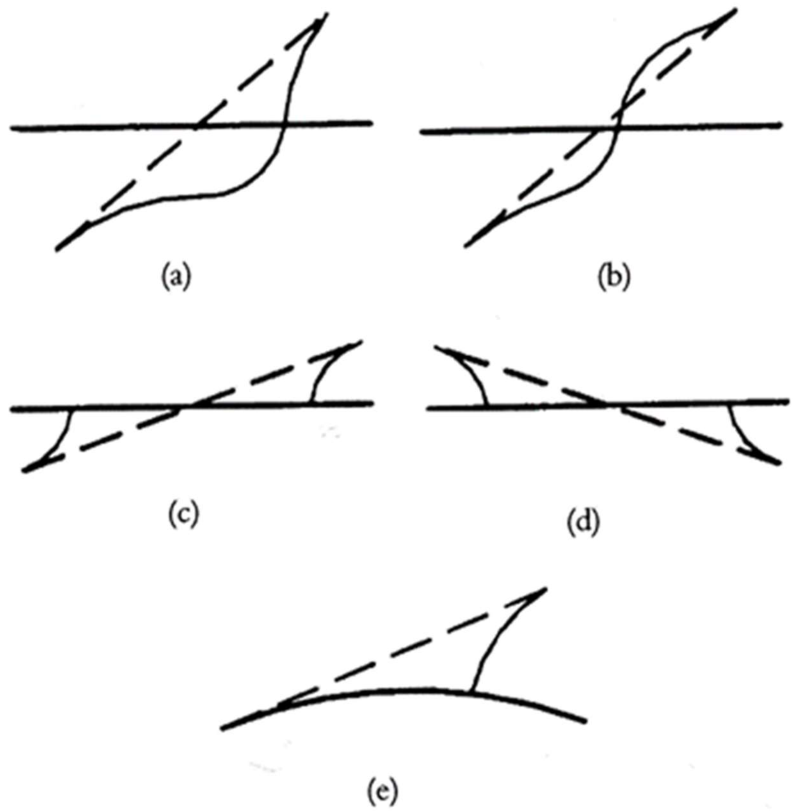


Figure 7.8 Alternative Methods of Realigning Skewed Intersections

- The angle of intersection of approaches should be approximately 90.

Problems with acute angle (Skewed) include

  - Visibility
  - Longer crossing times in some cases.
  - Larger pavement area.

Profile (Vertical)

- Should facilitate driver's control of vehicle.
- Avoid significant changes in grade.
- Typically, grade should be $\leq$ 3%
- Continue major street grade through intersection.
    - Approach (flat and straight as possible)
        - Avoid $>$6% on low speed ($<$40mph) and $>$ 3% on high speed ($\geq$ 50 mph)
    - Provide minimum grades and maximum vertical curve lengths.
    - Make adjustments away from the Intersection.
    - Traffic lanes should be visible and obvious to motorists.
    - Motorists should understand the path they are supposed to take.

Kerb Radius.

Factors governing the design of curves at grade intersection are

- Design vehicle
- Turning speed
- Traffic volume and composition

- Angle of turn.
- Approach width and parking
- Channelization
- Pedestrians

When the turning speed at intersection is less than 25 km/h, then the curve is designed to conform to at least the minimum turning path of the design vehicle.
If the speed greater than 25 km/h then the design speed is considered

Three types of design are used when turning speed less than 25 km/h

- Simple curve
- Simple curve with taper
- 3- centered compound curve (three simple curves)

Simple curve for PC

Radius should not be less than 7m for 90^0 right turn. Increase the radius to 10 m will increase the clearance from 20cm to 40 cm at the end of the curve. **(Figure 7.9 a)**

Simple curve with taper

The design is shown in **(figure 7.9b)** is a simple curve with tapers of 1:10 at each end and an offset 0.8 m with radius of 6 m

3- centered compound curve (three simple curves)

The layout of three- centered compound curve is shown in **(figure 7.9c)** this type of curve composed of three circular curves of radii of 30, 6, 30m

Table 7.2 Minimum Edge of Pavement Design for Turns at Intersections: Simple Curves and Simple Curves with Taper

Angle of Turn (degree)	Design Vehicle	Simple Curve Radius (ft)	Simple Curve Radius with Taper		
			Radius (ft)	Offset (ft)	Taper L:T
30	P	60	—	—	—
	SU	100	—	—	—
	WB-40	150	—	—	—
	WB-50	200	—	—	—
	WB-62	360	220	3.0	15:1
	WB-67	380	220	3.0	15:1
	WB-100T	260	125	3.0	15:1
	WB-109D	475	260	3.5	20:1
45	P	50	—	—	—
	SU	75	—	—	—
	WB-40	120	—	—	—
	WB-50	175	120	2.0	15:1
	WB-62	230	145	4.0	15:1
	WB-67	250	145	4.5	15:1
	WB-100T	200	115	2.5	15:1
	WB-109D	—	200	4.5	20:1
60	P	40	—	—	—
	SU	60	—	—	—
	WB-40	90	—	—	—
	WB-50	150	120	3.0	15:1
	WB-62	170	140	4.0	15:1
	WB-67	200	140	4.5	15:1
	WB-100T	150	95	2.5	15:1
	WB-109D	—	180	4.5	20:1
75	P	35	25	2.0	10:1
	SU	55	45	2.0	10:1
	WB-40	—	60	2.0	15:1
	WB-50	—	65	3.0	15:1
	WB-62	—	145	4.0	20:1
	WB-67	—	145	4.5	20:1
	WB-100T	—	85	3.0	15:1
	WB-109D	—	140	5.5	20:1
90	P	30	20	2.5	10:1
	SU	50	40	2.0	10:1
	WB-40	—	45	4.0	10:1
	WB-50	—	60	4.0	15:1
	WB-62	—	120	4.5	30:1
	WB-67	—	125	4.5	30:1
	WB-100T	—	85	2.5	15:1
	WB-109D	—	115	2.9	15:1

(Continued)

Table 7.2 Minimum Edge of Pavement Design for Turns at Intersections: Simple Curves and Simple Curves with Taper (*continued*)

Angle of Turn (degree)	*Design Vehicle*	*Simple Curve Radius (ft)*	*Simple Curve Radius with Taper*		
			Radius (ft)	*Offset (ft)*	*Taper L:T*
105	P	—	20	2.5	8:1
	SU	—	35	3.0	10:1
	WB-40	—	40	4.0	10:1
	WB-50	—	55	4.0	15:1
	WB-62	—	115	3.0	15:1
	WB-67	—	115	3.0	15:1
	WB-100T	—	75	3.0	15:1
	WB-109D	—	90	9.2	20:1
120	P	—	20	2.0	10:1
	SU	—	30	3.0	10:1
	WB-40	—	35	5.0	8:1
	WB-50	—	45	4.0	15:1
	WB-62	—	100	5.0	15:1
	WB-67	—	105	5.2	15:1
	WB-100T	—	65	3.5	15:1
	WB-109D	—	85	9.2	20:1
135	P	—	20	1.5	10:1
	SU	—	30	4.0	10:1
	WB-40	—	30	8.0	15:1
	WB-50	—	40	6.0	15:1
	WB-62	—	80	5.0	20:1
	WB-67	—	85	5.2	20:1
	WB-100T	—	65	5.5	15:1
	WB-109D	—	85	8.5	20:1
150	P	—	18	2.0	10:1
	SU	—	30	4.0	8:1
	WB-40	—	30	6.0	8:1
	WB-50	—	35	7.0	6:1
	WB-62	—	60	10.0	10:1
	WB-67	—	65	10.2	10:1
	WB-100T	—	65	7.3	10:1
	WB-109D	—	65	15.1	10:1
180	P	—	15	0.5	20:1
	SU	—	30	1.5	10:1
	WB-40	—	20	9.5	5:1
	WB-50	—	25	9.5	5:1
	WB-62	—	55	10.0	15:1
	WB-67	—	55	13.8	10:1
	WB-100T	—	55	10.2	10:1
	WB-109D	—	55	20.0	10:1

SOURCE: *A Policy on Geometric Design of Highways and Streets,* American Association of State Highway and Transportation Officials, Washington, D.C., 2004, pp. 584–587. Used with permission.

Table 7.3 Minimum Edge of Pavement Design for Turns at Intersections: Three-Centered Curves

		3-Centered Compound		*3-Centered Compound*	
Angle of Turn (degree)	*Design Vehicle*	*Curve Radii (ft)*	*Symmetric Offset (ft)*	*Curve Radii (ft)*	*Asymmetric Offset (ft)*
30	P	–	–	–	–
	SU	–	–	–	–
	WB-40	–	–	–	–
	WB-50	–	–	–	–
	WB-62	–	–	–	–
	WB-67	460-175-460	4.0	300-175-550	2.0-4.5
	WB-100T	220 -80-220	4.5	200- 80-300	2.5-5.0
	WB-109D	550-250-550	5.0	250-200-650	1.5-7.0
45	P	–	–	–	–
	SU	–	–	–	–
	WB-40	–	–	–	–
	WB-50	200-100-200	3.0	–	–
	WB-62	460-240-460	2.0	120-140-500	3.0-8.5
	WB-67	460-175-460	4.0	250-125-600	1.0-6.0
	WB-100T	250- 80-250	4.5	200- 80-300	2.5-5.5
	WB-109D	550-200-550	5.0	200-170-650	1.5-7.0
60	P	–	–	–	–
	SU	–	–	–	–
	WB-40	–	–	–	–
	WB-50	200- 75-200	5.5	200- 75-275	2.0-7.0
	WB-62	400-100-400	15.0	110-100-220	10.0-12.5
	WB-67	400-100-400	8.0	250-125-600	1.0-6.0
	WB-100T	250- 80-250	4.5	200- 80-300	2.0-5.5
	WB-109D	650-150-650	5.5	200-140-600	1.5-8.0
75	P	100- 25-100	2.0	–	–
	SU	120- 45-120	2.0	–	–
	WB-40	120- 45-120	5.0	120- 45-195	2.0-6.5
	WB-50	150- 50-150	6.5	150- 50-225	2.0-10.0
	WB-62	440- 75-440	15.0	140-100-540	5.0-12.0
	WB-67	420- 75-420	10.0	200- 80-600	1.0-10.0
	WB-100T	250- 80-250	4.5	100- 80-300	1.5-5.0
	WB-109D	700-125-700	6.5	150-110-550	1.5-11.5
90	P	100- 20-100	2.5	–	–
	SU	120- 40-120	2.0	–	–
	WB-40	120- 40-120	5.0	120- 40-200	2.0-6.5
	WB-50	180- 60-180	6.5	120- 40-200	2.0-10.0
	WB-62	400- 70-400	10.0	160- 70-360	6.0-10.0
	WB-67	440- 65-440	10.0	200- 70-600	1.0-11.0
	WB-100T	250- 70-250	4.5	200- 70-600	1.0-5.0
	WB-109D	700-110-700	6.5	100- 95-550	2.0-11.5

(*Continued*)

Table 7.3 Minimum Edge of Pavement Design for Turns at Intersections: Three-Centered Curves (*continued*)

Angle of Turn (degree)	*Design Vehicle*	*3-Centered Compound* *Curve Radii (ft)*	*3-Centered Compound* *Symmetric Offset (ft)*	*3-Centered Compound* *Curve Radii (ft)*	*3-Centered Compound* *Asymmetric Offset (ft)*
105	P	100- 20-100	2.5	—	—
	SU	100- 35-100	3.0	—	—
	WB-40	100- 35-100	5.0	100- 55-200	2.0-8.0
	WB-50	180- 45-180	8.0	150- 40-210	2.0-10.0
	WB-62	520- 50-520	15.0	360- 75-600	4.0-10.5
	WB-67	500- 50-500	13.0	200- 65-600	1.0-11.0
	WB-100T	250- 60-250	5.0	100- 60-300	1.5-6.0
	WB-109D	700- 95-700	8.0	150- 80-500	3.0-15.0
120	P	100- 20-100	2.0	—	—
	SU	100- 30-100	3.0	—	—
	WB-40	120- 30-120	6.0	110- 30-180	2.0-9.0
	WB-50	180- 40-180	8.5	150- 35-220	2.0-12.0
	WB-62	520- 70-520	10.0	80- 55-520	24.0-17.0
	WB-67	550- 45-550	15.0	200- 60-600	2.0-12.5
	WB-100T	250- 60-250	5.0	100- 60-300	1.5-6.0
	WB-109D	700- 85-700	9.0	150- 70-500	7.0-17.4
135	P	100- 20-100	1.5	—	—
	SU	100- 30-100	4.0	—	—
	WB-40	120- 30-120	6.5	100- 25-180	3.0-13.0
	WB-50	160- 35-160	9.0	130- 30-185	3.0-14.0
	WB-62	600- 60-600	12.0	100- 60-640	14.0-7.0
	WB-67	550- 45-550	16.0	200- 60-600	2.0-12.5
	WB-100T	250- 60-250	5.5	100- 60-300	2.5-7.0
	WB-109D	700- 70-700	12.5	150- 65-500	14.0-18.4
150	P	75- 20- 75	2.0	—	—
	SU	100- 30-100	4.0	—	—
	WB-40	100- 30-100	6.0	90- 25-160	1.0-12.0
	WB-50	160- 35-160	7.0	120- 30-180	3.0-14.0
	WB-62	480- 55-480	15.0	140- 60-560	8.0-10.0
	WB-67	550- 45-550	19.0	200- 55-600	7.0-16.4
	WB-100T	250- 60-250	7.0	100- 60-300	5.0-8.0
	WB-109D	700- 65-700	15.0	200- 65-500	9.0-18.4
180	P	50- 15- 50	0.5	—	—
	SU	100- 30-100	1.5	—	—
	WB-40	100- 20-100	9.5	85- 20-150	6.0-13.0
	WB-50	130- 25-130	9.5	100- 25-180	6.0-13.0
	WB-62	800- 45-800	20.0	100- 55-900	15.0-15.0
	WB-67	600- 45-600	20.5	100- 55-400	6.0-15.0
	WB-100T	250- 55-250	9.5	100- 55-300	8.5-10.5
	WB-109D	700- 55-700	20.0	200- 60-500	10.0-21.0

SOURCE: *A Policy on Geometric Design of Highways and Streets,* American Association of State Highway and Transportation Officials, Washington, D.C., 2004, pp. 588–591. Used with permission.

Kerb Radius.

- General guidance
    - 10 to 25ft local
    - 25 to 30ft collectors
    - 30 to 35 ft un-channelized intersections with arterials.

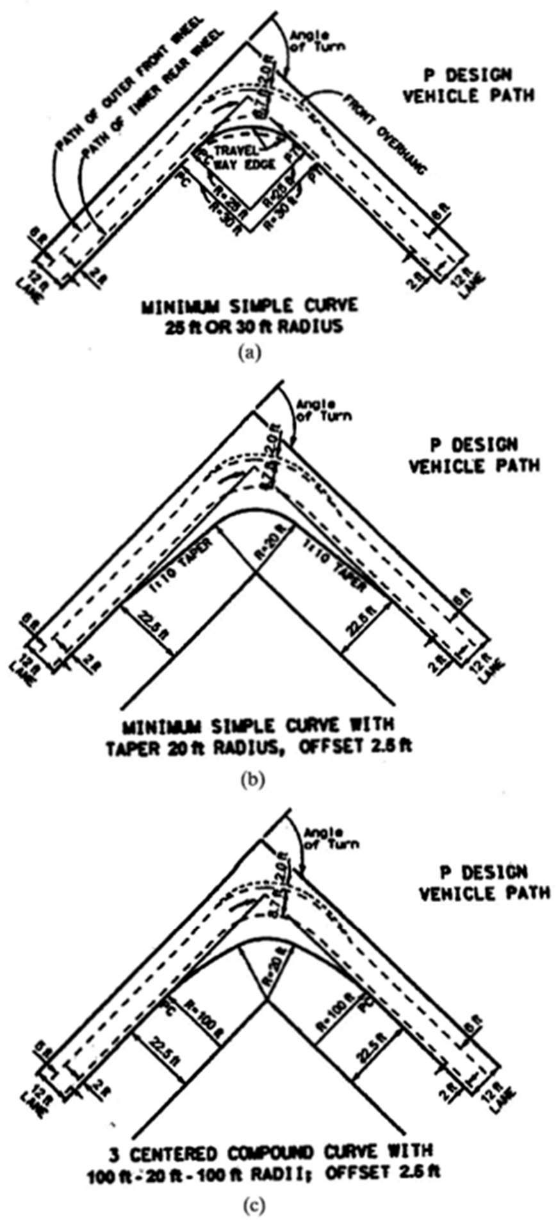


Figure 7.9 Minimum Designs for Passenger Vehicles

SOURCE: *A Policy on Geometric Design of Highways and Streets,* American Association of State Highway ar Transportation Officials, Washington, D.C., 2004, p. 595. Used with permission.

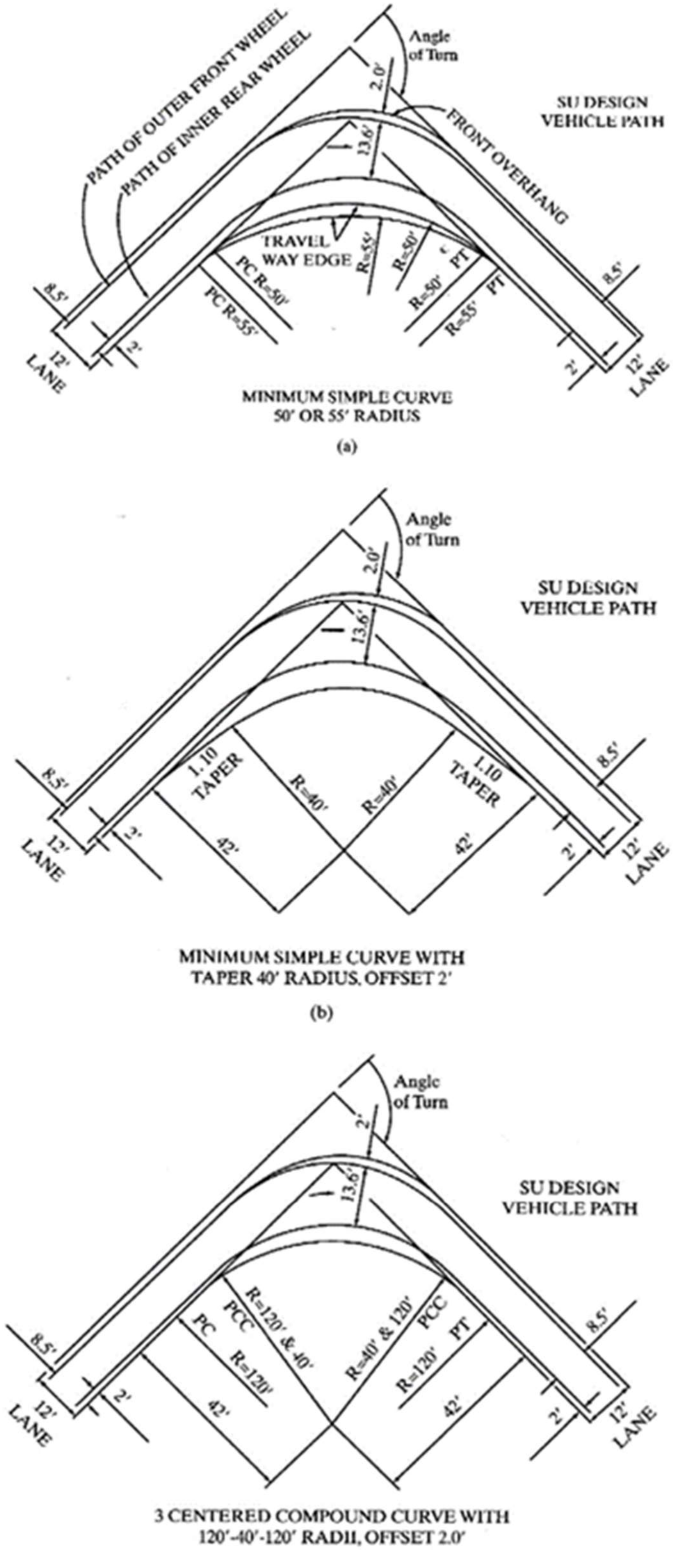


Figure 7.10 Minimum Design Curves for Single Unit Trucks and City Transit Buses

SOURCE: *A Policy on Geometric Design of Highways and Streets,* American Association of State Highway and Transportation Officials, Washington, D.C., 2004, p. 597. Used with permission.

Channelization.

Channelization at an intersection is used to achieve one or more of the following objectives.

- Separates conflicting movements.
- Controls merging, diverging, and crossing angle of vehicles.
- Decreases vehicle wander.
- Provides clear path for different movements.
- Gives priority to dominant movements
- Provide pedestrian refuge
- Provides storage area for turning vehicles
- Controls prohibit turns.
- Restricts speed
- Provide space for traffic control devices.

Sight Distance Requirements for No-Control Intersections – Case A

- In this situation, the intersection is not controlled by a yield sign, stop sign, or traffic signal.
- sight distance should be provided for vehicle approaching the intersection from either side to see a crossing vehicle and if necessary to adjust the vehicle's speed so as to avoid a collision.
- This distance must include the distance travelled by the vehicle both during the driver's perception reaction time and during brake actuation or the acceleration to regulate speed.
- AASHTO has suggested that a driver may take up to 2.5 seconds to detect and recognize a vehicle at intersections
- Also, AASHTO has noted that field observations have indicated that drivers tend to decrease their speeds to about 50 percent of their mid-block speed as they approach intersections that have no control.
- Based on this information, ASSHTO has suggested the distances shown in Table 7.7 for different approach speeds. Figure 7.20 shows a schematic of the sight triangle required for the location of an obstruction that will allow for the provision of the minimum distance d_a and d_b.
- These minimum distances depend on the approaching speed as shown in Table 7.7.

For example, if a road with a speed limit of $64\ km/h$ intersects with a road with a speed limit of $40\ km/h$, the distances d_a and d_b are 60m and 35m, respectively. It should be noted that the distances shown in Table 7.7 allow time for drivers to come to a stop before reaching the intersection.
However, it should be noted that the distances tend to be lower than those given in chapter 3 for the corresponding speeds because of the phenomenon of drivers reducing their speeds as they the approach intersections with no controls.
It can be seen from figure 7.20 that triangles ABC and ADE are similar, which gives

$$\frac{CB}{AB} = \frac{ED}{AD} \tag{7.3}$$

and

$$\frac{d_b}{d_a} = \frac{a}{d_a - b} \tag{7.4}$$

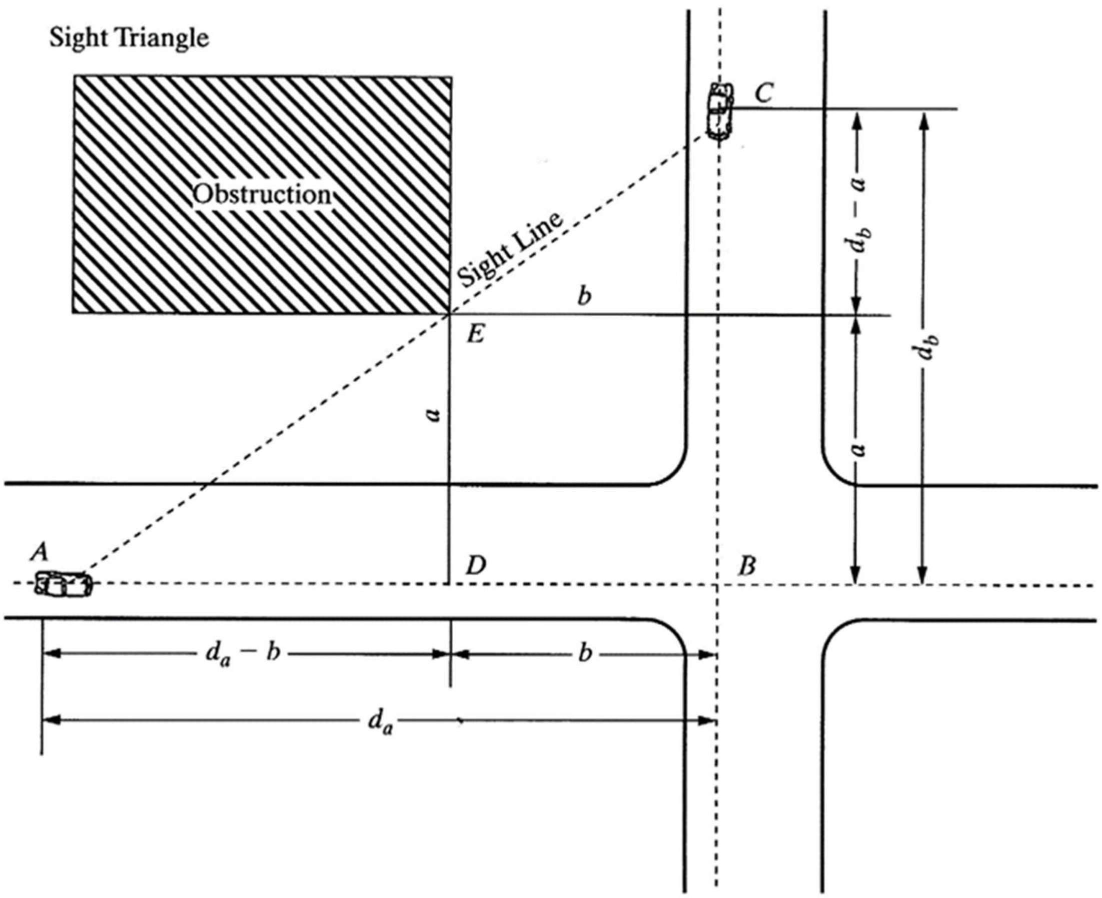


Figure 7.20 Minimum Sight Triangle at a No-Control or Yield Control Intersection Cases A and C.

Suggested Lengths and Adjustments of Sight-Triangle Leg Case A – No Traffic.

Design Speed (mi/h)	Length of Leg (ft)
15	70
20	90
25	115
30	140
35	165
40	195
45	220
50	245
55	285
60	325
65	365
70	405
75	445
80	485

If any three of the variables $d_{a,} d_{b,}$ a and b are known, the fourth can be determined using equation 7.4.

Example 7.2 Computing Speed Limit on a Local Road.
A tall building is located 14m from the centerline of the right lane of local road (b in figure 7.20) and 20m from the centerline of the right lane of an intersecting road (a in figure 7.20). If the maximum speed limit on the intersecting road is 56km/h, what should the speed limit on the local road be such that the minimum sight distance is provided to allow drivers of approaching vehicles to avoid imminent collision by adjusting their speeds? Approach grades are 2%.

Solution:

- Determine the distance on the local road at which the driver first sees traffic on the intersecting road.
  Speed limit on intersecting road $= 56km/h$
  Distance required on intersecting road (($d_a) = 50m$ (from table 7.7)
  Calculate the distance available on local road by using Equation 7.4

$$d_b = a\frac{d_a}{d_a - b}$$
$$= 20\frac{50}{50 - 14}$$
$$= 28m$$

- Determine the maximum speed allowable on local road.

The maximum speed allowable on local road is 32km/h (from table 7.7).
No correction is required for the approach grade as it is less than 3%.

Grade Separations.

A grade separation is the arrangement of taking one road over or under another by means of a bridge. (also known as a fly-over junction.) or an under pass.

Underpass: an underpass or tunnel is an underground passageway completely closed except the openings for entrance and exit at each end.

Overpass: it is also known as a flyover that cross over another road or railway.

The grade separation and interchanges may be warranted:

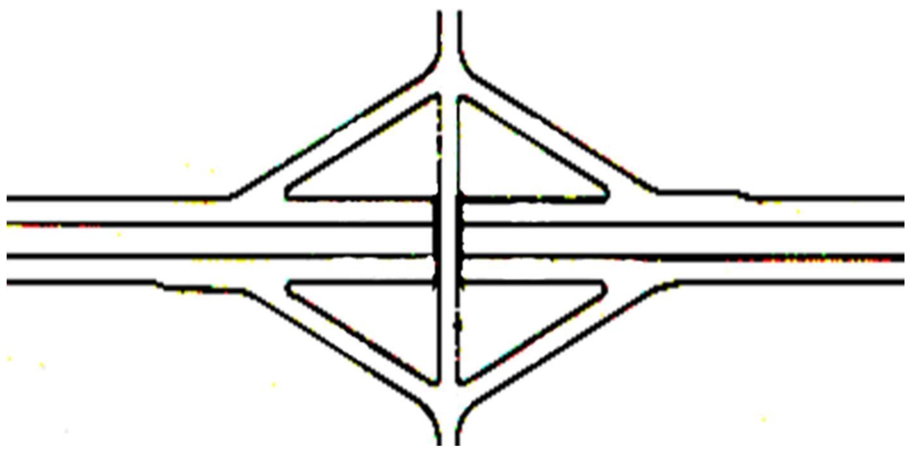
1. If it was decided to have limited access cross roads.
2. To eliminate bottlenecks.
3. To prevent accidents and increase safety if there was a high number of accidents at the junction.
4. If the topography is such that the other types of design are not feasible.
5. If there was a high traffic volume.

Grade separated:

The interchanges may be of various types. The most common types of interchanges are:

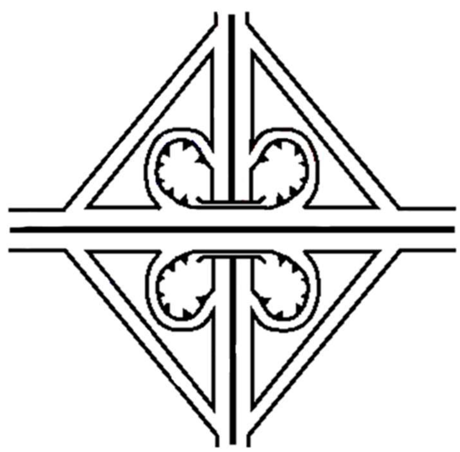
Diamond interchange:

1 It is the simplest
2 Most common four leg interchange.
3 Least costly type of interchange
4 Off ramp from the freeway terminates with an at-grade intersection at the minor road.
5 Not expensive
6 Little right of way required.



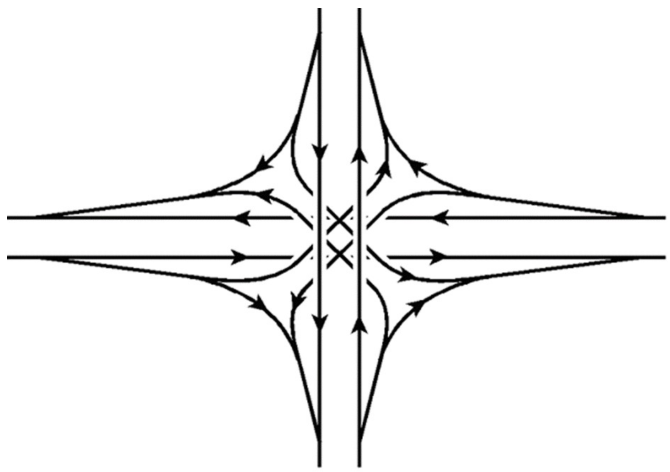
Cloverleaf interchange:

1. This is also a four-legged interchanged.
2. It is used when two highways of high volume and speed intersect with each other.
3. It consists of slip ramps and loop ramps.
4. No at grade intersection (unlike diamond interchange) therefore it can be used to connect two controlled access roads.
5. Higher right of way than diamond.
6. The main advantage of a cloverleaf intersection is that it provides complete separation of traffic. Also, high speed at intersections can be achieved.
7. The disadvantage is that a large area of land is required.



All Directional interchange

1. It provides direct travel towards destination
2. High speed
3. High capacity
4. No indirect movement
5. Used for freeway to freeway connection
6. Very high cost because of multiple level.

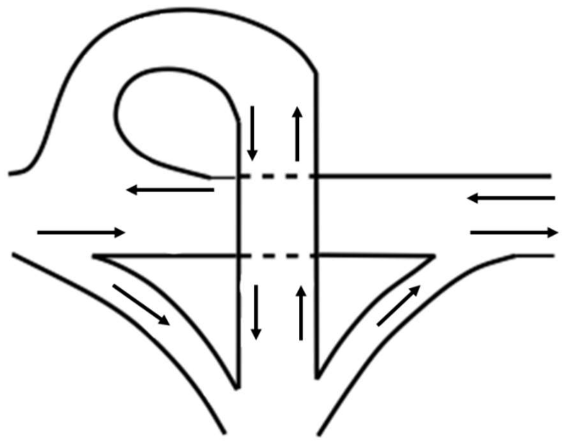


Trumpet interchange:

This is a three-legged interchange used as T-intersection.

Little right of way.

Inexpensive.



Highway Drainage

Water is the major factor which contribute to the failure of highway. therefore, adequate drainage is the most important consideration in locating and designing highways and city streets

- A means by which surface water is removed from pavement and ROW.
- Redirects water into appropriately designed channels.
- Eventually discharges into natural water systems.

Rural Drainage System

This system includes:

- travers and longitudinal slopes.
- Longitudinal channels (side ditches)
- Culverts and bridges.

Urban Drainage System (City Streets)

This system includes:

- travers and longitudinal slopes.
- Underground pipe drains.
- Curbs and gutters
- Inlets, catch basins and manholes.

Inadequate Drainage will result in;

- Damage to highway structures.
- Loss of capacity.
- Visibility problems with spray of water.
- Safety problems, reduced friction.

There are two types of drainage

1. **Surface drainage**.
   This is removing the water from the highway pavement (water coming from rain and snow).
2. **Subsurface drainage**.
   This is to control ground water, this can be a problem where water table is near the pavement structure.

Rural Surface Drainage.

- **Transverse Slopes**.
    - Removes water from pavement surface, this is achieved by crowning the surface of the centre of the pavement
    - Facilitated by cross-section elements (cross-slope, shoulder slope)

      Cross slope of 2% does not affect driver comfort.

- **Longitudinal Slopes**
    - Minimum gradient to maintain adequate slope in longitudinal channels.
    - Slop should not be less than 0.2% in flat terrain.
    - Zero slope may be used on uncurbed pavement with adequate cross slope.

- **Longitudinal Channel (side ditches)**
  - Ditches along side of the road to collect surface water run-off.
  - They are found in rural areas in cut sections.
  - They are V-shaped and flat bottomed. (flat bottomed are preferred)
  - Avoid deep and narrow ditches.

Urban Surface Drainage. (Drainage of City Streets)

- The surface drainage of city streets is different from rural highways.
- Open channels cannot be used,
  1. they are unsightly
  2. occupy more space
  3. serve as a source of danger to traffic and pedestrians.
  4. Therefore, underground drains are used.

The surface water drainage in city streets composed of the following:

- **Pavement Crown (cross slope)**

  The pavement crown should be as small as possible for appearance and safety.

- **Longitudinal Slopes.**

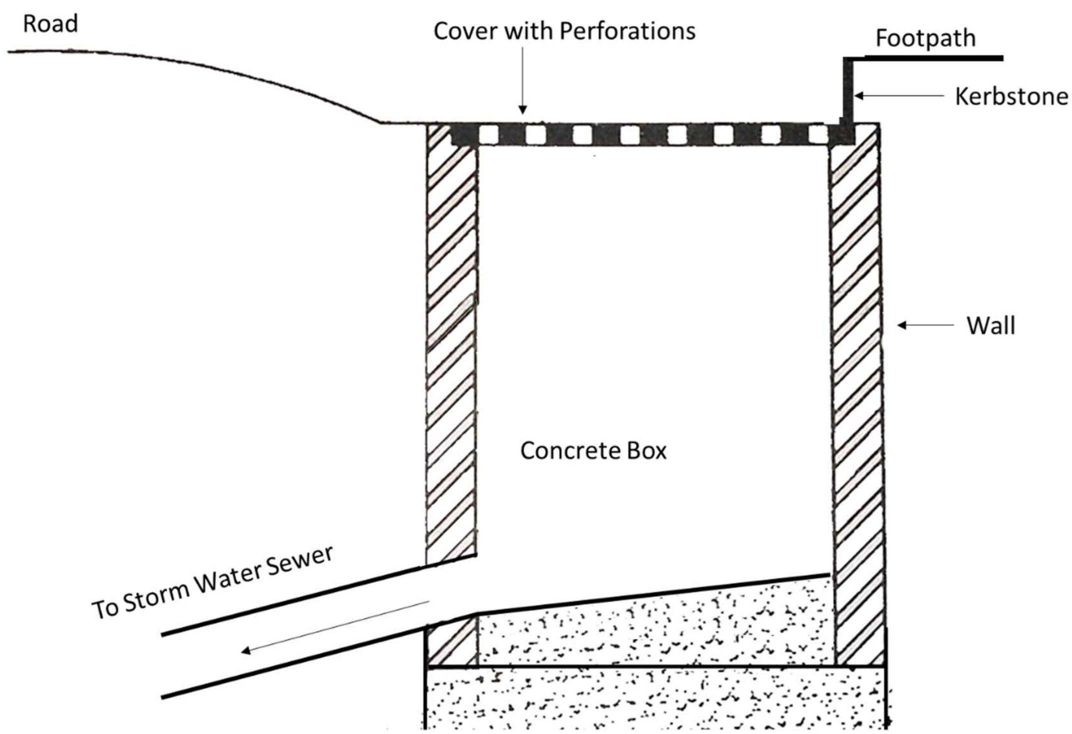
- **Kerb and Gutters.**

  Kerb and gutters can be used to control drainage in addition to other functions which include preventing the encroachment of vehicles on adjacent areas and delineating pavement edges.

- **Inlets**

  They are usually provided at intersection by the side of road to intercept water flowing in gutters. Inlets are connected by manholes; the inlet is simply a concrete box it may have opening in vertical direction or in horizontal direction. Inlets have only an outlet pipe placed at the bottom of the inlet.

  They are subjected to clogging at the opening so they must be cleaned frequently.

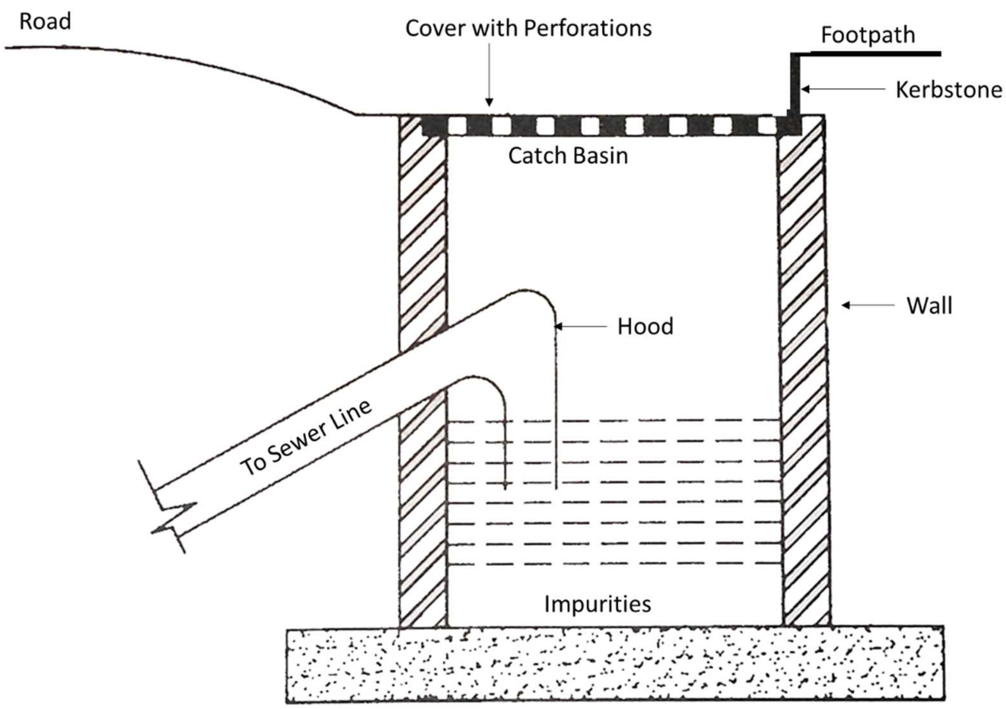


Inlet with Horizontal Opening

- **Catch Basins.**

  They are similar to inlets in their function and design, the difference is the outlet pipe is placed at some distance above the bottom of the chamber.

  The purpose of the catch basin is that debris flushed from the street is trapped in the bottom of the catch basin so that it does not enter the storm sewer. They need good maintenance, if silt build up is not removed they function as inlet



Catch Basin

- **Manholes.**

Storm sewer system are subjected to partial or complete clogging and facilities must be provided for cleaning.

Manholes are placed at points where;

The sewer change direction

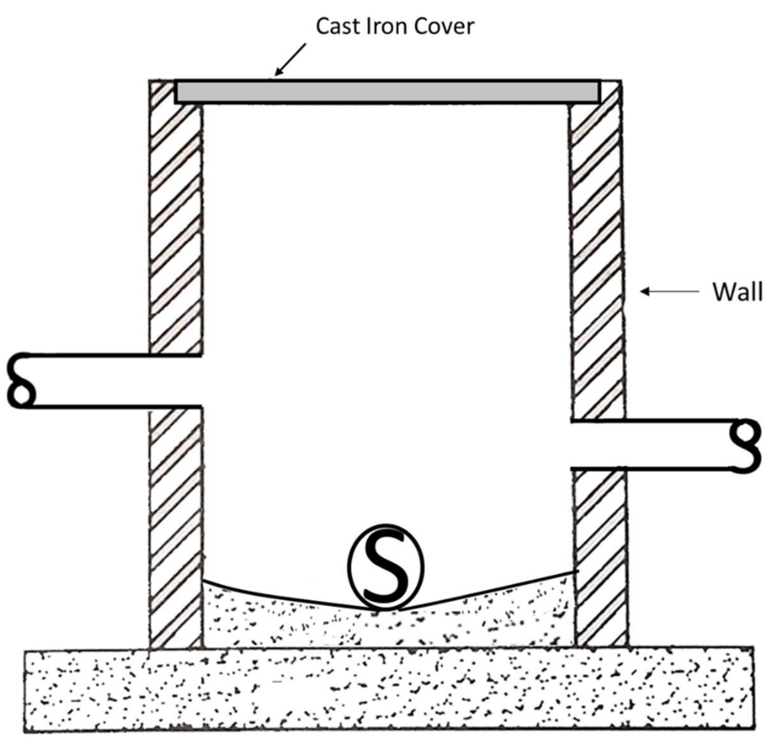
The sewer change grade

Junction are made

And intermediates points (90 - 150 m).

The opening should be large enough to permit a man to enter the chamber in which he can work (1.2 m) diameter.

Manhole covers are cast iron circular shape 0.6 m diameter.



Manhole

Drainage Structures.

Drainage structures are constructed to carry over natural waterways that flow below the right of way of the highway .
They also provide the flow of water below the highway along the natural channel without disturbing its course.
One of the main concern is always to provide adequate size structure (opening is sufficiently large to discharge expected flow of water).

Types of Drainage Structures.

1 Major Structures (bridges).

2 Minor Structures (culverts).

The difference between culvert and bridges is the span length
If the span length < 20 ft (6 m) is called culvert.
Span length > 20ft (6m) is called bridge.

Culverts

Location:

Culverts are found in three locations.

1 at the bottom of depressions
2 where natural stream intersects the roadway.
3 At location required for passing surface drainage carried in side ditch.

Alignment:

A culvert should be laid on a straight alignment which may be either.

1 cross road at right angle.
2 Or skewed to the road centre line or it should be aligned with the natural flow of water.
3 Culvert if possible should cross at right angle to reduce cost.

Grade

A slope of 2% is sufficient to keep culvert clean and keeping the water speed at reasonable level.

1 increasing the grade will increase the water speed, this will cause erosion therefore protective measures are needed at the outlet.
2 If the grade is reduced, then velocity may be reduced. This will cause sediments in the water deposited in the culvert, therefore capacity is reduced.

Shapes:

Circular, Box, Elliptical, Arch, and pipe arch.
They are made of concrete or corrugated metal.

Circular and box culvert.

Operating conditions:

There are two flow conditions are possible in culvert flow

1. Inlet controlled, this is where culvert behave as open channel
2. Out let controlled, this is where the culvert behaves as pipe under full conditions

Culvert design procedure

The design procedure involves the following.

- Obtain all site data and plot a roadway cross section at the culvert site, including a profile of the stream channel
- Establish the culvert elevations inlets and outlets and determine culvert length and slope.
- Select type and size of culvert
- Examine the need for energy dissipaters.

Design of Surface Drainage System

The design of drainage system is divided into three major Phases.

1. An estimate of the quantity of water to reach the system.
2. The hydraulic design of each element of the system.
3. The comparison of alternative system. (take the lowest cost)

Hydrologic Analysis.

- Estimating of the quantity of water.

The rational run-off method is used to estimate run-off from drainage area. The rational formula is given by the equation:

$$Q = 0.0028 \quad CIA_d$$

Q = The peak discharge (m^3/sec)
C = Coefficient representing ratio or runoff to rainfall.
I = Intensity of rainfall (mm/hour)
A_d = Drainage area (hectares =10000 m^2)

Culvert cross section area

The following equation can be used to estimate the size of culvert needed for a peak discharge

$$Q = \quad VA_c$$

Where:

Q = The peak discharge (m^3/sec)
V= the water speed in culvert in m/s
A_c = the cross-sectional area of the culvert in m^2

Run-off Coefficient, C

The design of drainage must be provided for all rainfall does not infiltrate the soil.
The value of run-off coefficient, C depends on the type of ground cover, the slope of the drainage area and the moisture content.

The table below shows Runoff Coefficient for the Rational Formula

Description of drainage Area	Runoff Coefficients
Pavement, Portland cement, or asphalt Pavement gravel surface	0.75 – 0.95 0.250 – 0.60
Urban residential areas. Rural residential areas Urban business district.	0.50 – 0.70 0.35 – 0.60 0.60 – 0.80
Sandy soil, cultivated Clay soil cultivated	0.15 – 0.30 0.30 – 0.75
Parks, golf courses grassy meadows	0.15 – 0.30

Drainage area

The drainage area A_d is the total area from which the surface water is expected to flow. This area can be determined from contour maps or by studying the topography of the drainage area.

In case the drainage area consists of different ground characteristic with different runoff coefficient C_1, C_2, C_3 and their respective areas A_1, A_2, A_3 the weighted average value of C_w is calculated by using the following equation:

$$C_w = \frac{A_1C_1 + A_2C_2 + A_3C_3}{A_1 + A_2 + A_3}$$

Rainfall intensity (I).

In order to determine the peak discharge of water, information about the rainfall intensity, I, is needed.

Some storms provide light rainfall for only a few minutes whereas others provide heavy rain for extended periods of time.

For design purposes, the quantity of water considered is a function of the duration of the storm, its intensity (i.e., rainfall in mm/h), and the probability of storm occurrence.

Storm occurrence probabilities are accounted for by using the return period.

The return period and storm probability are reciprocals. For example, a peak water discharge having a 50-year return period, has an occurrence probability of 0.02. A 50-year return period has a 2% chance of being equalled or exceeded in any given year. Likewise, a 100-year return period has a 1% chance of being equalled or exceeded in any given year. The higher the return period, the more costly it is.

Type of Highway Facility	Return Period (Storm Frequency) in Years
Interstate Highways	50
Limited Access Freeways & Arterials	25
Collectors	10
Locals	10
City Streets	10

Table 4.13 Storm Frequency for Highway Culvert Design

Time of Concentration (T_c)

- Time for water to flow from most distant point on the drainage area to the point of interest within the drainage area.

Time of concentration depends on:

- Size and shape of drainage area.
- Type of surface.
- Slope of drainage area.
- Rain intensity.

$$T_c = \frac{L}{3600\,V}$$

Where:

$$T_c = travel\ time\ for\ section\ i\ in\ drainage\ area\ (hr)$$
$$L = flow\ length\ (m)$$
$$V = average\ runoff\ water\ velocity\ over\ surface\ (m/sec)$$

The runoff speed of surface water is given in the table below.

Average Runoff Water Speed Over Surface Conditions for Time of Concentration (In Meters per Second).

	Slope (%)						
Surface Condition	0-3	4-7	8-10	11-15	16-20	21-25	26-30
Woodland	0.15	0.30	0.45	0.50	0.60	0.80	1010
Pastureland	0.25	0.45	0.65	0.80	0.90	1.25	1.35
Cultivated (row crop)	0.30	0.60	0.90	1.10	1.20	1.35	1.50
Pavement	1.50	3.65	4.70	5.50	-	-	-

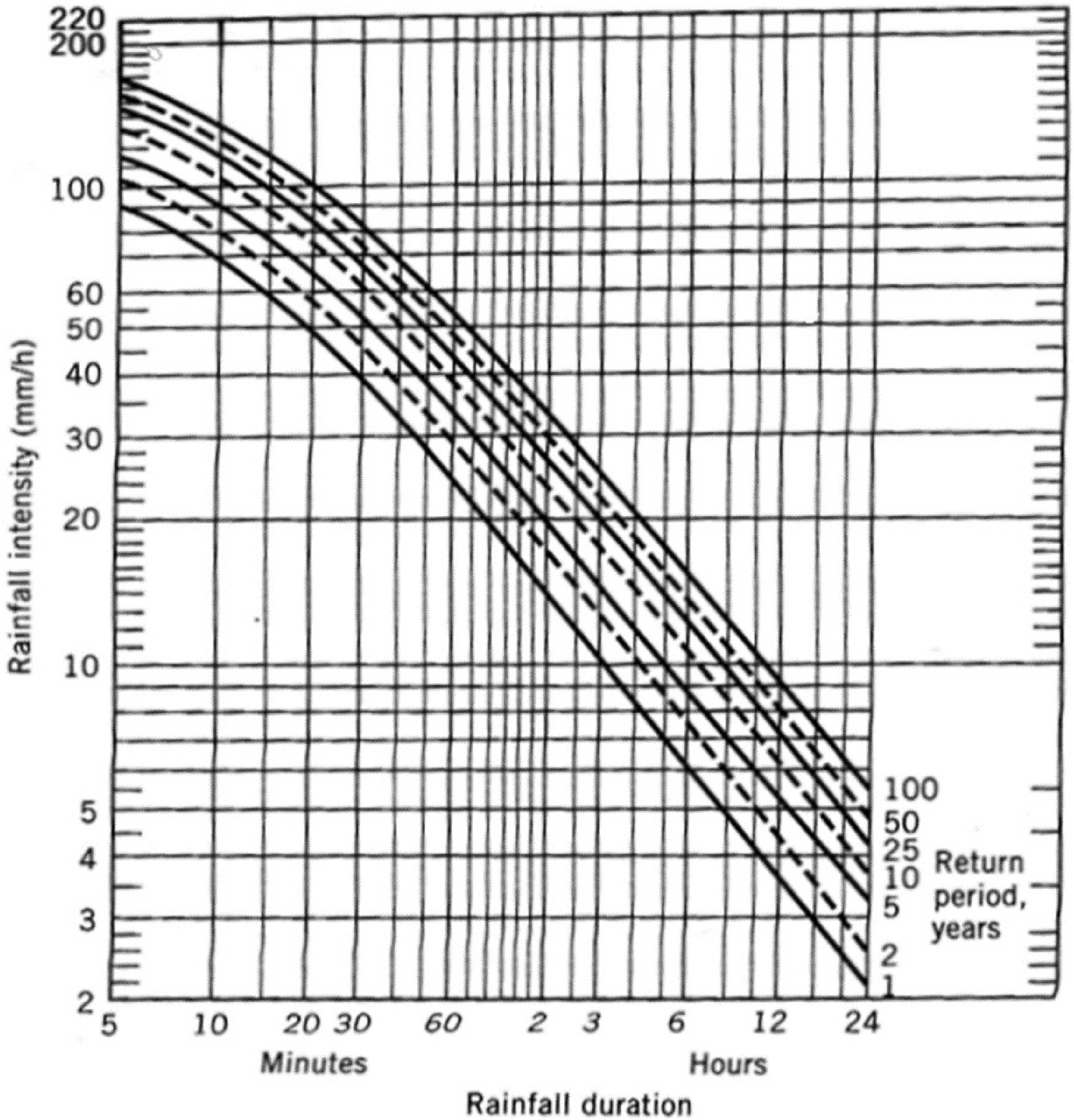


Figure Rainfall intensity –rainfall duration curves

Example:

An engineer plans to install a culvert under a collector – type highway to reduce flooding in the area. the drainage area is 9 hectares of pastureland in a rural residential area in central Pennsylvania. Water in the drainage area flows on an approximate 5% slope for 1215m before reaching the culvert. Estimate the peak discharge that can be expected at the culvert.

Solution:

The speed of the water for a 5% slope is 0.45 m/s (From the Table Average Runoff Water Speed Over Surface). Therefore, time of concertation is

$$\frac{1215m}{0.45m/s} = 2700s = 45min$$

Using the time of concentration as the required rainfall duration and the fact that a collector roadway require 10 years return period. Use the rainfall - intensity figure, to find a rainfall intensity of 47mm/h. In rural residential areas, a runoff coefficient of 0.35 to 0.60 is suggested (see runoff coefficient table). For maximum runoff (i.e., a conservative design), a factor of 0.60 is selected.
Using the discharge equation, the peak discharge at the culvert sight is:

$$\begin{gathered} Q = 0.00278(CIA) \\ = 0.00278(0.60)(47)(9) \\ = 0.706m^3/s \end{gathered}$$

Example 4.8

In the previous example, the peak discharge was found to be 0.706 m^3/s. It has been determined that a water flow speed of 1.5 m/s is needed to maintain scour velocity. Calculate the approximate diameter of a circular culvert needed to carry 0.706m^3/s if the culvert is flowing full. Assume that the flow in the culvert is not complex and water does pond at either end of the culvert.

Solution:

Using the equation 4.22, the cross-sectional area needed is:

$$A_c = \frac{Q}{V} = \frac{0.706}{1.5} = 0.471m^2$$

Converting the area of the culvert to a diameter, d,

$$d = 2\sqrt{\frac{A_c}{\pi}} = 2\sqrt{\frac{0.471}{3.1416}} = \underline{0.774m}$$

Because culvert pipe is only available in even increments, a 0.8m diameter pipe would be used.

Highway Drainage Ditch Design

The majority of water coming from highway facilities flows in open channels that run parallel to the highway.
As the size of the ditch increases, the complexity of the design also increases.
Besides the need to understand water flow in the ditches, there is a need to ensure the roadside is safe for errant vehicles. Cross slopes, channel shape, guiderail, and other safety features must also be evaluated for a complete design.

Table 4.15 Roughness Coefficient, n, for Manning's Formula.

Type of Channel Lining	n
Smooth Concrete	0.012
Smooth Asphalt	0.015
Earth	0.020
Rock	0.035
Grass and Brush	0.050
Ductile Iron Pipe	0.013
Corrugated Steel Pipe	0.024
Corrugated Plastic Pipe	0.024

As it is shown in Equation , $\boldsymbol{Q = VA_c}$

The capacity of a drainage facility is a function of the cross-sectional area and the speed of the water. The area is easy to obtain from the geometric shape and dimensions of the facility. The speed of the water is determined from the classic equation known as Manning's Formula. The equation applies to steady flow in a uniform open channel. Manning's formula is written as:

$$V = \frac{R_h^{2/3} S^{1/2}}{n}$$

Where:

V = the water speed in m/s,
S = the slope of the channel in m/m,
n = Manning's roughness coefficient (values of n are given in table 4.15).
R_h =the hydraulic radius,

$$R_h = \frac{A_c}{WP}$$

Where A_c is the area of cross-section in m^2, and WP is the wetted perimeter (the wetted length in the cross section in meters). The hydraulic elements of various cross sections (i.e., area, wetted perimeter, and hydraulic radius) are presented in table 4.16.

Example 4.9

A highway agency plans to construct a rock-lined drainage channel next to a highway. The channel will be rectangular in shape with a 1.0m bottom and a water depth of 0.5m. The slope of the channel is determined to be 3.0%. Calculate the speed of the water and the flow rate.

Solution:

The area of flow in the channel is

$$A_c(1.0)(0.5) = 0.5m^2$$

The wetted perimeter, WP, is

$$WP = 1.0 + (2)(0.5) = 2.0m$$

The hydraulic radius, R_h, is calculated with Equation 4.24 and is

$$R_h = \frac{0.5}{2} = 0.25m$$

Table 4.15 shows that Manning's roughness coefficient for a rock-lined channel is 0.035. Applying Manning's formula (Eq. 4.23) gives

$$V = \frac{R_h^{2/3} S^{1/2}}{n}$$

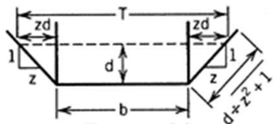
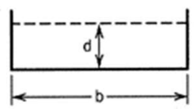
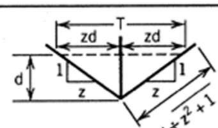
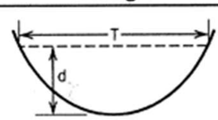
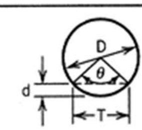
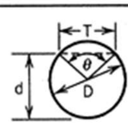
$$= \frac{(0.25)^{2/3}(0.03)^{1/2}}{0.035}$$

$$= \underline{1.98m/s}$$

Using Equation 4.22, the flow rate is

$$Q = VA_c = 1.98(0.5) = \underline{0.99m^3/s}$$

Table 4.16 Hydraulic Elements of Various Cross Sections

Cross Section	Area ($\mathscr{A}_c$)	Wetted Perimeter ($\mathscr{W}P$)	Hydraulic Radius ($\mathscr{R}_h$)
 Trapezoid	$bd + zd^2$	$b + 2d\sqrt{z^2+1}$	$\dfrac{bd + zd^2}{b + 2d\sqrt{z^2+1}}$
 Rectangle	bd	$b + 2d$	$\dfrac{bd}{b + 2d}$
 Triangle	zd^2	$2d\sqrt{z^2+1}$	$\dfrac{zd}{2\sqrt{z^2+1}}$
 Parabola	$\frac{2}{3}dT$	$T + \dfrac{8d^2}{3T}$ 1	$\dfrac{2dT^2}{3T^2 + 8d^2}$
 Circle < 1/2 full 2	$\dfrac{D^2}{8}\left(\dfrac{\pi\theta}{180} - \sin\theta\right)$	$\dfrac{\pi D\theta}{360}$	$\dfrac{45D}{\pi\theta}\left(\dfrac{\pi\theta}{180} - \sin\theta\right)$
 Circle > 1/2 full 3	$\dfrac{D^2}{8}\left(2\pi - \dfrac{\pi\theta}{180} + \sin\theta\right)$	$\dfrac{\pi D(360-\theta)}{360}$	$\dfrac{45D}{\pi(360-\theta)}\left(2\pi - \dfrac{\pi\theta}{180} + \sin\theta\right)$

1. Satisfactory approximation for the interval $0 < \dfrac{d}{T} \leqq 0.25$

When $\dfrac{d}{T} > 0.25$, use $WP = \dfrac{1}{2}\sqrt{16d^2 + T^2} + \dfrac{T^2}{8d}\sinh^{-1}\dfrac{4d}{T}$

2. $\theta = 4\sin^{-1}\sqrt{d/D}$
3. $\theta = 4\cos^{-1}\sqrt{d/D}$

Insert θ in degrees in equations

Design Steps.

1. The return period such as 10 years, 25 years, 50 years etc. is decided based on the finance available and desired margin of safety, for the design of drainage system.
2. The values of coefficient of run off from drainage area are found and the weighted value is computed.
3. The time for the flow of storm water from the farthest point in the drainage area to culvert inlet is estimated from the distance, slope of the ground and type of cover.
4. From the rain fall intensity – duration frequency curves the rain intensity is found in mm/h. corresponding to duration T and frequency of return period.
5. The total area of drainage A is found in units of 10,000 meter-square.
6. The quantity Q is computed = CIA.
7. The cross-section area of flow of the culvert is calculated = Q/V, where V is the allowable speed of flow in the culvert.