Transportation Depth Topics

School of PE

Refresher Notes

Code: CITER-D **Spring 2015**

Table of Contents

Also Attached:

- **HCM Primer**
- **MUTCD Primer**
- **AASHTO Roadside Design Guide Primer**
- **AASHTO Green Book Primer**

REFERENCES

Please make sure you have the following manuals while attending 24 Hour Depth classes for the Transportation subject:

- *Civil Engineering Reference Manual for the PE Exam*, 14th Edition (13th Edition is also fine), by Michael R. Lindeburg (Popularly known as CERM)
- *Highway Capacity Manual* (HCM 2010), 2010 edition, Transportation Research Board—National Research Council, Washington, DC. (Popularly known as HCM)
- *Manual of Uniform Traffic Control and Devices for Streets and Highways*, 2009, by FHWA (Popularly known as MUTCD)
- *A Policy on Geometric Design of Highways and Streets,* 2011, by AASHTO (Popularly known as AASHTO Green Book)
- *Roadside Design Guide*, 4th edition, 2011, by AASHTO

NOTE:

Please note that some topics in the Transportation Depth exam are related to other areas such as Construction, Geotechnical, and Water Resources. Those topics are covered in the regular 56 Hour Fundamentals Review. In the exam, if you see problems from other areas, please make sure you use the regular 56 Hour Fundamentals Review notes.

1. Surveying

NOTE: This section is reviewed under Geometrics section of the regular classes also. This is repeated here to have a quick overview before going into depth problems in the later sections.

1.1 Bearings and Azimuths

See definitions on CERM* page 78-12, section 23.

PROBLEM 1 - Convert the following bearings to azimuths from north:

(a) S 52^0 31' 18" W
(b) N 68^0 22' 54" W

SOLUTION 1

(a) $180^0 + 52^0$ 31' 18" = 232^0 31' 18"
(b) $360^0 - 68^0$ 22' 54" = 291^0 37' 06"

*NOTE: All references to CERM are to the 14th Edition of the Civil Engineering Reference Manual for PE Exam, by Michael R. Lindeburg

1.2 Latitude and Departures

- Latitude of a line is the distance that the line extends in a north or south direction. A line that runs towards north has a positive latitude; a line that runs towards south has a negative latitude.
- Departure of a line is the distance that the line extends in an east or west direction. A line that runs towards east has a positive departure; a line that runs towards west has a negative departure.

  See figure 78.8 on CERM Page 78-13.

*NOTE: All references to CERM are to the 14th Edition of the Civil Engineering Reference Manual for PE Exam, by Michael R. Lindeburg

PROBLEM 2
A traverse line that is 940.79 ft long has a bearing of S 23^o W. Determine:

i) the latitude of traverse line
A) -367.60 B) +367.60 C) -866.00 D) +866.00

ii) the departure of traverse line
A) -367.60 B) +367.60 C) -866.00 D) +866.00

SOLUTION 2
i) -940.79 * Cos 23^o = - 866.00 **(Answer C)**
ii) -940.79 * Sin 23^o = - 367.60 **(Answer A)**

1.3 Polar Coordinate System

It is based on Northing (N) and Easting (E)

PROBLEM 3
Determine the coordinates of point B on line AB, given the following:

Length of AB = 50'
Bearing of line AB = S 60^o 00' E
Coordinates of point A = N 322,000 and E 450,500

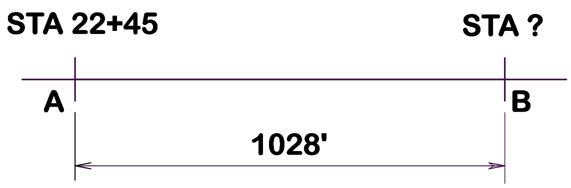
SOLUTION 3
Latitude of line AB = - 50' Cos 60^o = -25.0
Departure of line AB = 50' Sin 60^o = 43.3
Northing (N) of point B = 322,000 – 25 = N 321,975
Easting (E) of point B = 450,000 + 43.3 = E 450,043.3

1.4 Stationing

- Stationing concept is used along horizontal alignments for referencing purpose
- 1 station = 100 feet
- How do you represent stationing?
  - Specific location is represented as Sta 10+00
  - Distance is represented as 10.00 sta

PROBLEM 4

What is the station at Point B?



SOLUTION 4

Station at Point A = Sta 22+45

Station at Point B = (Sta 22+45) + 1028’ = 2245’+1028’ =3273’
= Sta 32+73

2. Driver Performance and Behavior

2.1 Information Processing and Perception

The time required to respond successfully to any driving situation, such as an emergency, involves four stages:

- perception (detection and identification)
- decision
- reaction
- response of the vehicle.

2.2 Driver Expectancy

A fundamental component of driver information processing and perception. Drivers operate with a set of expectancies, e.g.:

- freeway exits will be on the right side of the road (or the left in Britain, Australia, etc.);
- advance warning will be given of hazards in the roadway;
- other drivers will obey traffic regulations, etc.

2.3 Perception-Reaction Time

A significant variable in the successful processing and use of information is the speed with which this is done. Perception-Reaction Time (PRT) is a human factor often cited by traffic engineers concerned with safety. PRT is "the interval between the appearance of some object or condition in the driver's field of view and the initiation of a response" such as braking or changing course. Note that PRT involves the initiation of a response (e.g. pressing the brake), not the completion of the vehicle maneuver (stopping).

PRT depends on the situation. Response time is generally quickest when there is one specific response to be made to a single stimulus (brake lights of vehicle ahead). In the case of "choice reaction time," in which there is more than one stimulus and/or more than one possible response (e.g. toll plaza), reaction time increases as a function of the number of possibilities. A driver may, for example, have to decide whether to steer or brake, or both, to avoid a pedestrian.

The PRT used for design standards by AASHTO includes 1.5 sec for perception and decision, 1.0 sec for making a response, for a total of 2.5 sec, which is generally considered adequate for all but the most complex driving situations.

PROBLEM 5

Which of the following factors affect driver performance and behavior?

A. cell phone use
B. fatigue
C. traffic
D. drugs and alcohol
E. young / old age
F. law enforcement
G. All of the above

SOLUTION 5

The correct answer is G. All of the above affect driver performance and behavior.

2.4 Sight Distance

See Chapter 3 in "A Policy on Geometric Design of Highways and Streets" 2011 by AASHTO. This book is popularly known as Green Book

Sight distance is the length of roadway ahead that is visible to the driver. It relates to stopping, steering, and overtaking. There are four types of sight distance:

- Stopping Sight Distance (SSD)
- Decision Sight Distance (DSD)
- Passing Sight Distance (PSD)
- Intersection Sight Distance (ISD)

2.4.1 Stopping Sight Distance

Stopping Sight Distance is the sum of two distances: (1) the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied; and (2) the distance needed to stop the vehicle from the instant the brake application begins. These are referred to as *brake reaction distance* and *braking distance*, respectively.

The AASHTO GB provides the following equations for calculating braking distance and SSD, with and without the effect of grades.

The following equation includes terms for both the *brake reaction distance* and *braking distance* (Use CERM equation 79.43(b) on page 79-10)*:*

$$SSD = 1.47V_{mph}t + \frac{V_{mph}^2}{30\left(\left(\frac{a}{32.2}\right) \pm G\right)}$$

where:

SSD = Stopping Sight Distance, ft;
V = design speed, mph;
t = breaking reaction time, 2.5 sec
a = deceleration rate, 11.2 ft/sec^2
G = percent of grade divided by 100, it is in decimal

NOTE: $\frac{a}{32.2}$ can be replaced with f as discussed on CERM page 79-10, section 14

First part of the equation represents *brake reaction distance* and the second part represent *braking distance.*

If a vehicle is traveling on a level roadway, then SSD can be determined using the following simplified equation:

$$SSD = 1.47V_{mph}t + 1.075\,\frac{V_{mph}^2}{a}$$

2.4.2 Braking or Skidding Distance

If the vehicle does not come to a full stop, then the following equation can be used to calculate braking or skidding distance:

$$D = \frac{V_0^2 - V^2}{30\left(\frac{a}{32.2} \pm G\right)}$$

Where

V_0 = Initial Speed in mph

V = Final Speed in mph

NOTE: $\frac{a}{32.2}$ can be replaced with friction factor $'f'$, $if\ f'$ is given.

PROBLEM 6

A motorist is traveling on a level grade at 50 mph. A tree has fallen across the road and forces the motorist to stop. Assuming a 2.5 sec PRT and 11.2 ft/sec^2 deceleration rate, determine the *brake reaction distance* and *braking distance* in feet.

A. 147' and 154'
B. 165' and 194'
C. 184' and 240'
D. 165' and 290'
E. 184' and 194'

SOLUTION 6

$$\text{Brake reaction distance} = 1.47Vt = 1.47(50)(2.5) = \underline{\underline{184'}}$$

$$\text{Braking distance} = 1.075\,\frac{V^2}{a} = 1.075\,\frac{50^2}{11.2} = \underline{\underline{240'}}$$

The correct answer is C, 184' and 240'

NOTE: CERM Table 79.2 on page 79-11 shows AASHTO minimum SSDs for various design speeds.

PROBLEM 7

A motorist is traveling down a 6% grade at 65 mph and needs to stop because of a crash scene. Assuming a 2.0 sec PRT and 12.0 ft/sec^2 deceleration rate, determine the total SSD in feet.

SOLUTION 7

$$\text{SSD} = 1.47Vt + \frac{V^2}{30\left(\left(\frac{a}{32.2}\right) \pm G\right)} = 1.47(65)2 + \frac{65^2}{30\left(\left(\frac{12}{32.2}\right) - 0.06\right)}$$

$$= 191.1 + \frac{4225}{9.380} = \underline{\underline{641.52'}}$$

PROBLEM 8
Determine approximate Minimum Stopping Sight Distance required on a bicycle path, given the following data:

Velocity, V	= 25 mph
Coefficient of friction, f	= 0.25
Grade , G	= + 6%

(A) 10 ft (B) 78 ft (C) 88 ft (D) 160 ft

SOLUTION 8

$$S = 1.47\,Vt + \frac{V^2}{30(f \pm G)}$$

Where:

S = Stopping Sight Distance (ft).
V = Design speed (mph) = 25 mph
G = Grade (ft/ft) (rise/run) = + 6%
f = Coefficient of friction = 0.25
T = Perception Reaction Time = 2.5 sec

Apply the above equation:

$$S = 1.47 * 25 * 2.5 + \frac{25^2}{30(0.25 + 0.06)} = 158.95ft$$

Therefore the correct answer is D.

PROBLEM 9
An unexpected obstacle occurred on a freeway with 65 mph speed limit. All drivers travelling on this freeway have to stop. An alter driver who requires a perception reaction time of 2.5 seconds travelling at the posted speed limit. An impaired driver who requires a perception reaction time of 3.5 seconds travelling at 70 mph speed. How much additional distance the impaired driver would be travelling over the alert driver before the brakes are applied?

SOLUTION 9

Using CERM equation 79.43(b),

Brake Reaction Distance in feet = $1.47V_{mph}t$

Brake Reaction Distance travelled by the alert driver = 1.47 * 65 * 2.5 = 239 ft

Brake Reaction Distance travelled by the impaired driver = 1.47 * 70 * 3.5 = 360 ft

Additional distance travelled by the impaired driver = 360 – 239 = 121 ft

2.4.3 Decision Sight Distance

Decision Sight Distance (DSD) is the distance needed for a driver to detect an unexpected or otherwise difficult-to-perceive information source or condition in a roadway environment that may be visually cluttered, recognize the condition, or its potential threat, select an appropriate speed and path, and initiate and complete the avoidance maneuver safely and efficiently. DSD's increase with design speed and the complexity of the situation encountered. DSD's are tabulated in the AASHTO GB for following five avoidance maneuvers:

Table 3-3. Decision Sight Distance

Metric						U.S. Customary					
Design Speed (km/h)	Decision Sight Distance (m)					Design Speed (mph)	Decision Sight Distance (ft)				
	Avoidance Maneuver						Avoidance Maneuver				
	A	B	C	D	E		A	B	C	D	E
50	70	155	145	170	195	30	220	490	450	535	620
60	95	195	170	205	235	35	275	590	525	625	720
70	115	325	200	235	275	40	330	690	600	715	825
80	140	280	230	270	315	45	395	800	675	800	930
90	170	325	270	315	360	50	465	910	750	890	1030
100	200	370	315	355	400	55	535	1030	865	980	1135
110	235	420	330	380	430	60	610	1150	990	1125	1280
120	265	470	360	415	470	65	695	1275	1050	1220	1365
130	305	525	390	450	510	70	780	1410	1105	1275	1445
						75	875	1545	1180	1365	1545
						80	970	1685	1260	1455	1650

Avoidance Maneuver A: Stop on rural road—t = 3.0 s

Avoidance Maneuver B: Stop on urban road—t = 9.1 s

Avoidance Maneuver C: Speed/path/direction change on rural road—t varies between 10.2 and 11.2 s

Avoidance Maneuver D: Speed/path/direction change on suburban road—t varies between 12.1 and 12.9 s

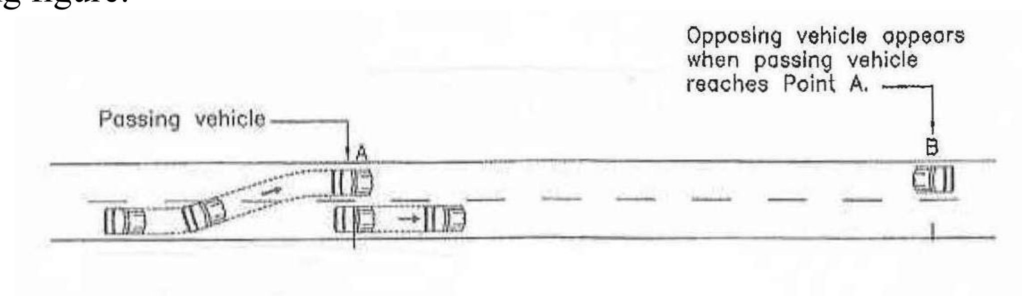
Avoidance Maneuver E: Speed/path/direction change on urban road—t varies between 14.0 and 14.5 s

PROBLEM 10
What is the required decision sight distance (DSD) in the following situation: You are driving along a winding rural road at 40 mph when you are about to approach a one lane bridge that can allow vehicles to go in one direction at a time.
A. 330 ft. B.690 ft. C.600 ft. D.715 ft.

SOLUTION 10
This situation requires Avoidance Maneuver C, therefore the DSD = 600 ft. If no vehicle is seen approaching a stop in opposite direction, you may proceed across the bridge with caution.

2.4.4 Passing Sight Distance

Passing Sight Distance (PSD) is applicable on two-lane highways as shown in the following figure:



PSD values are tabulated below:

Table 3-4. Passing Sight Distance for Design of Two-Lane Highways

Metric				U.S. Customary			
Design Speed (km/h)	Assumed Speeds (km/h) Passed Vehicle	Assumed Speeds (km/h) Passing Vehicle	Passing Sight Distance (m)	Design Speed (mph)	Assumed Speeds (mph) Passed Vehicle	Assumed Speeds (mph) Passing Vehicle	Passing Sight Distance (ft)
30	11	30	120	20	8	20	400
40	21	40	140	25	13	25	450
50	31	50	160	30	18	30	500
60	41	60	180	35	23	35	550
70	51	70	210	40	28	40	600
80	61	80	245	45	33	45	700
90	71	90	280	50	38	50	800
100	81	100	320	55	43	55	900
110	91	110	355	60	48	60	1000
120	101	120	395	65	53	65	1100
130	111	130	440	70	58	70	1200
				75	63	75	1300
				80	68	80	1400

SOURCE: AASHTO Green Book

PROBLEM 11
Two vehicles are heading north on a level two-lane highway. The driver in the lead car is trying to conserve gas and is driving at the 50 mph speed limit. The driver of the following car is running late, is trying to make up for lost time by traveling at 60 mph, and intends to overtake and pass the lead car. What is the estimated passing sight distance required in this situation.

A. 700’
B. 800’
C. 900’
D. 1000’

SOLUTION 11
The correct answer is D, 1000 ft., as indicated in the 2011 AASHTO Green Book, Table 3-4.

2.4.5 Intersection Sight Distance

Detailed discussion on this topic is presented under Intersection Design section.

3. Horizontal Curves

NOTE: Some portions of this section are reviewed under Geometrics section of the regular classes also. Necessary fundamental topics are repeated here to have a quick overview before going into depth problems.

3.1 Circular Curves

- A horizontal circular curve is an arc between two straight lines known as tangents.

  See CERM equations 79.1 to 79.10 and Figure 79.1 on pages 79-2 and 79-3

- Stationing on Horizontal Curves can be determined using CERM equations 79.11 and 79.12 on page 79-3.

PROBLEM 12

For the following circular curves having radius R, what is their degree of curve by Arc definition and Chord definition?

SOLUTION 12

(a) Roadway curve with 500.00 ft

$$D_a = \frac{5729.578'}{500\,ft} = 11^0 27'33''$$

(b) Railroad curve with 500.00 ft

$$D_c = 2(\sin^{-1})\left(\frac{50}{500\,ft}\right) = 11^0 28'42''$$

PROBLEM 13

Compute the following for a Highway curve with R = 750.000 ft, I = 18^0 30', and PI Sta 123+24.80:

D, T, L, E, HSO, and stations of the PC and PT

SOLUTION 13

$$D_a = \frac{5729.578'}{750\,ft} = 7^\circ 38' 22''; \qquad I = 18^\circ + 30'/60' = 18.5^\circ;$$

$$T = R \times tan\left(\frac{I}{2}\right) = 750 \times tan\left(\frac{18.5^\circ}{2}\right) = 122.145\,ft$$

$$L = \frac{I}{360^\circ}\,2\pi R = \frac{18.5^\circ}{360^\circ}\,2\pi 750 = 242.164\,ft$$

$$E = R \times tan\left(\frac{I}{2}\right) tan\left(\frac{I}{4}\right) = 750 \times tan\left(\frac{18.5^\circ}{2}\right) tan\left(\frac{18.5^\circ}{4}\right)$$

$$= 750 \times 0.1629 \times 0.0809 = 9.881\,ft$$

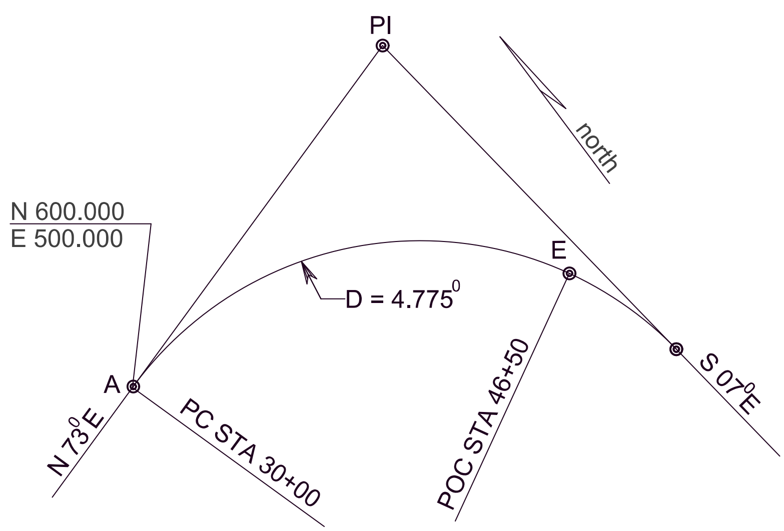
$$HSO = R\left(1 - \cos\frac{I}{2}\right) = 750\left(1 - \cos\frac{18.5^\circ}{2}\right) = 750(0.013) = 9.753\,ft$$

$$PC\ Sta. = PI\ Sta. - T = 12324.800 - 122.145 = 122 + 02.655$$

$$PT\ Sta. = PC\ Sta. + L = 12202.655 + 242.164 = 124 + 44.819$$

PROBLEM 14

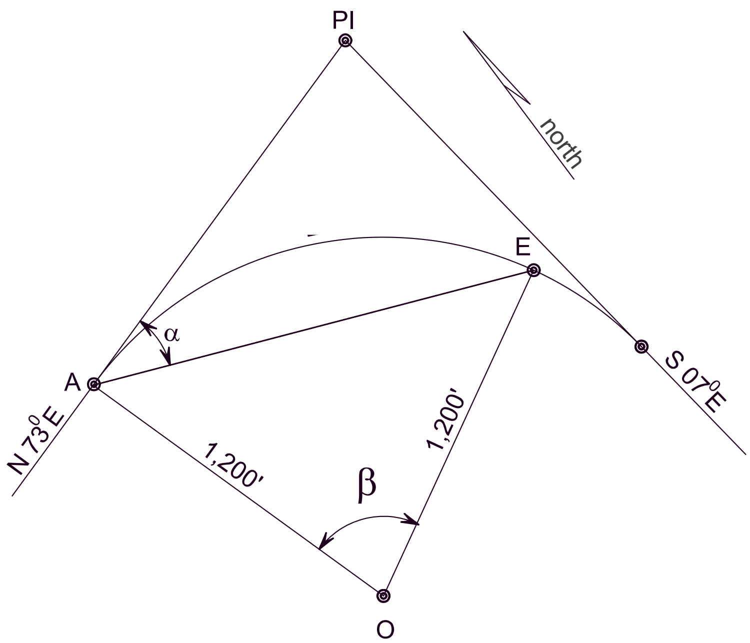
Given the horizontal curve shown, determine the coordinates of Point E.



SOLUTION 14

Step 1. Establish Chord A-E.
Step 2. Find Bearing and Length of Chord A-E.
Step 3. Use Latitudes and Departures to Determine Coordinates of Point E.

Step 1. Establish Chord $\overline{A - E}$.



Find Radius. $R = \frac{5729.578}{D} = \frac{5729.578}{4.775^0}$
$= 1{,}200.000'$

Find Arc Length A-E
= Sta 46+50 – Sta 30+00 =1,650 ft.

Establish Chord $\overline{A\text{-}E}$.

Find Angle β.

Use Angle β to find Angle α.

Note that α is $\frac{\beta}{2}$.

Use Angle α to find the Bearing of Chord $\overline{A\text{-}E}$.

Step 2. Find Bearing and Length of Chord $\overline{A - E}$

Notice that Triangle A-O-E has two sides of length R separated by angle β.

$$\text{Angle } \beta = \frac{1{,}650'}{2\pi R} \times 360^0 = \frac{825'}{\pi(1{,}200')} \times 360^0 = 78.782^0 \quad \therefore \text{ Angle } \alpha = \frac{78.782^0}{2} = 39.391^0$$

Bearing of Chord $\overline{A\text{-}E}$ = Back Tangent Bearing + Angle α = N 73^0 E + 39.391^0 = **S 67.609^0 E**

$$\text{Length of Chord } \overline{A\text{-}E} = 2 \times 1{,}200\left(\sin\frac{78.782^0}{2}\right) = 2(1{,}200)\sin 39.392^0 = \mathbf{1523.06'}$$

Step 3. Use Latitudes and Departures of Chord $\overline{AE}$ to Determine Coordinatesof Point E.

Chord $\overline{AE}$ has a SE bearing. Therefore the Latitude is negative and the Departure is positive.

Latitude is change in north-south direction:

$$= \text{Length} \times \cos(\text{Bearing Angle}) = 1523.06' \times \cos 67.61^0 = -580.147'$$

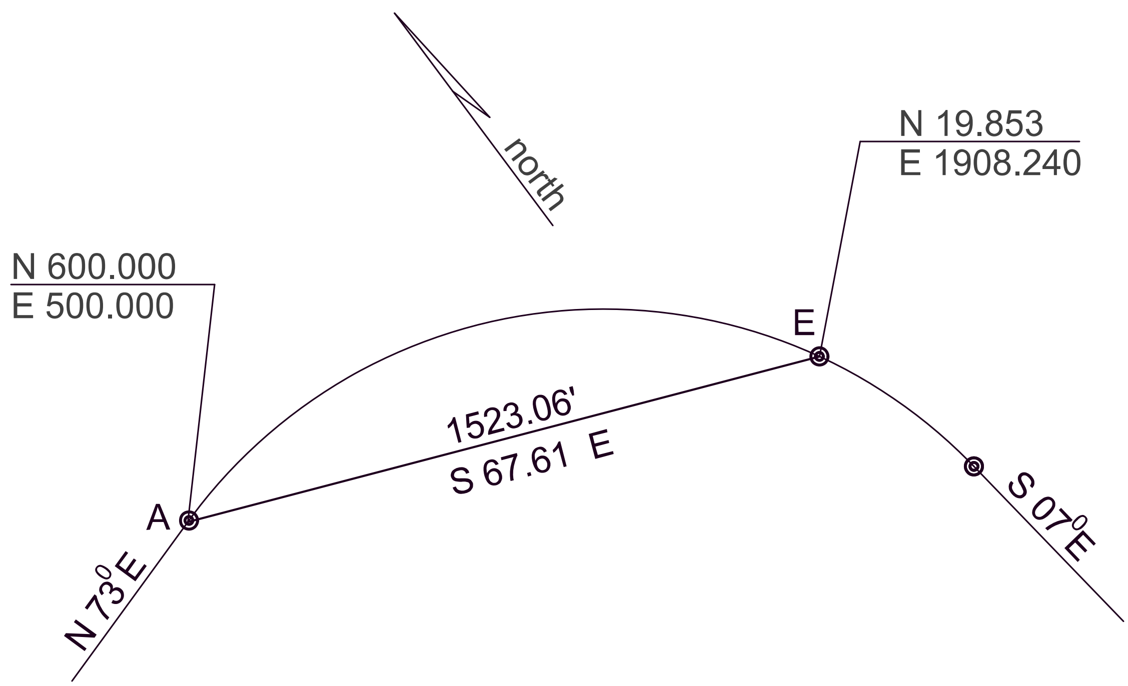
Departure is change in east-west direction:

$$= \text{Length} \times \sin(\text{Bearing Angle}) = 1523.06' \times \sin 67.61^0 = +1408.240'$$

Coordinates of Point E

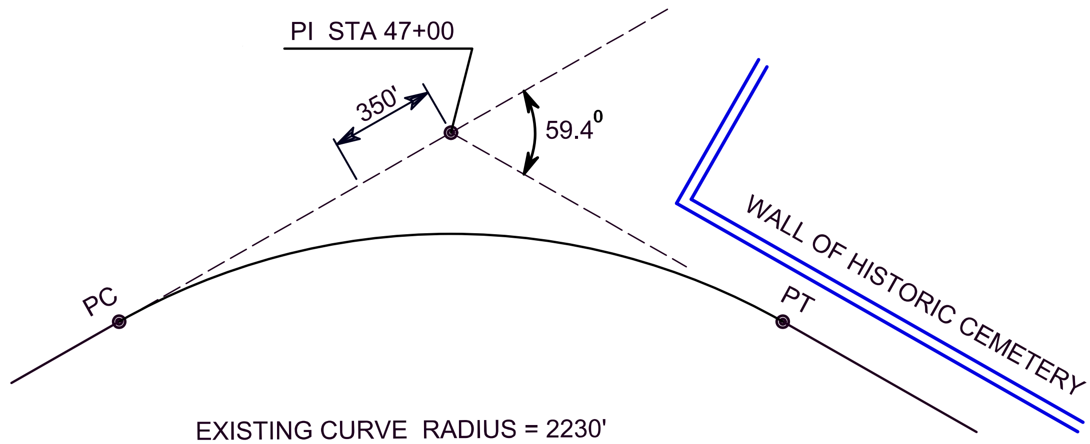
Point E N – coordinate + Latitude: 600.000 – 580.147' = **N 19.853**

Point E E – coordinate + Departure: 500.000 + 1,408.240' = **E 1, 908.240**



PROBLEM 15

The centerline of an existing road is located near a historic cemetery as shown. The PT of the curve is too close to the wall of the cemetery for the proposed roadway widening. The existing curve has a PI located at Sta. 47+00, a deflection angle of 59.4^0 and a radius of 2230'. The roadway centerline will be shifted 350 ft along the back tangent to avoid the cemetery. The existing PC and deflection must remain the same. The radius (ft) of the new curve is most nearly:



SOLUTION 15

Determine the tangent length of the existing curve.

$T_{exist} = R_{exist}$ tan I/2 = 2230 tan 59.4/2 = 1271.97'

Note that the tangent length of the new curve is 350' shorter than T_{exist}.

$T_{new} = T_{exist}$ - 350'= 1271.97'- 350' = 921.97'

Solve for the radius of the new curve using T_{new}.

T_{new} = 921.97'= R_{new} tan I/2 = R_{new} tan 59.4/2

Therefore R_{new} = 921.97' / (tan 59.4/2) = 1616'

3.2 Superelevation

- Used at horizontal curves
- Use CERM equation 79.37(b) on page 79-7

PROBLEM 16

What is the minimum radius, R_{min}, that can be used on a horizontal curve with a 70 mph design speed, a maximum superelevation, e_{max} = 0.08, and a side friction factor, f = 0.10?

SOLUTION 16

Use CERM equation 79.37(b);

e_{max} = 8%; V = 70 mph; f_{max} = 0.10

$$R_{min} = \frac{V_{mph}^2}{15(e_{max} + f_{max})}$$

(NOTE: In AASHTO GB Eqn 3-8, e_{max} is multiplied with 0.01. If you use AASHTO Eqn, then e_{max} should be in Percent. If you use CERM eqn, e_{max} should be in fraction. Both should give the same answer)

$$R_{min} = \frac{70^2}{15(0.08 + 0.10)} = 1814.80 \text{ ft}$$

See AASHTO Green Book (2011) Table 3-7 for e_{max} = 8%, page 3-32.

PROBLEM 17

A proposed bicycle racetrack will have several horizontal curves, each with a design speed of 30 mph. The track will be superelevated at 2% through the curves, and will have a paved surface with a coefficient of friction that varies from 0.31 at 12 mph to 0.21 at 30 mph. What is the minimum curve radius appropriate for this racetrack?

(A) 156 ft (B) 225 ft (C) 260 ft (D) 294 ft

SOLUTION 17

The equation for minimum radius:

$$R = \frac{V^2}{15(e+f)} = \frac{30^2}{15(0.02 + 0.21)} = 260\ ft$$

Therefore the correct answer is C.

3.3 Superelevation Transitions

- To change from normal crown to superelevation and vice versa
- Transitions – See CERM page 79-8

PROBLEM 18

A two lane highway curves to the right. The lanes are each 12-ft wide. The following information is provided:

Design Speed	60 mph
Normal cross slope	2.0% or 0.02 ft/ft
Superelevation	6.2% or 0.062 ft/ft
Superelevation runoff rate	1:222

The superelevation is to be developed by rotating the pavement about the roadway centerline and using 2/3 and 1/3 rule. Use the above information to determine:

a) Tangent runout
b) Superelevation runoff

SOLUTION 18

See CERM Figure 79.8

a) Tangent runout, T_R = wp/SRR (CERM Equation 79.40)
= 12(0.02)/(1/222) = 53.28 ft

b) Superelevation runoff, L = we/SRR (CERM Equation 79.41)
= 12(0.062)/(1/222) = 165.17 ft

NOTE: If SRR is not given, refer to the AASHTO Green Book Table 3-17. The table shows L values based on the design speed.

3.4 Stopping Sight Distance on Horizontal Curve Section

Obstructions along the inside of curves can limit the available (chord) sight distance. A curve must be designed that will simultaneously provide the required stopping sight distance while maintaining a clearance from a roadside obstruction.

See CERM equation 79.45 and figure 79.9 on page 79-11.

PROBLEM 19

A 6-lane divided highway has a design speed of 65 mph and the following typical section:

Grassed median:	14 ft wide
Median shoulders:	4 ft wide
Lanes:	12 ft wide, 3 lanes directional
Rightside shoulders:	10 ft. wide

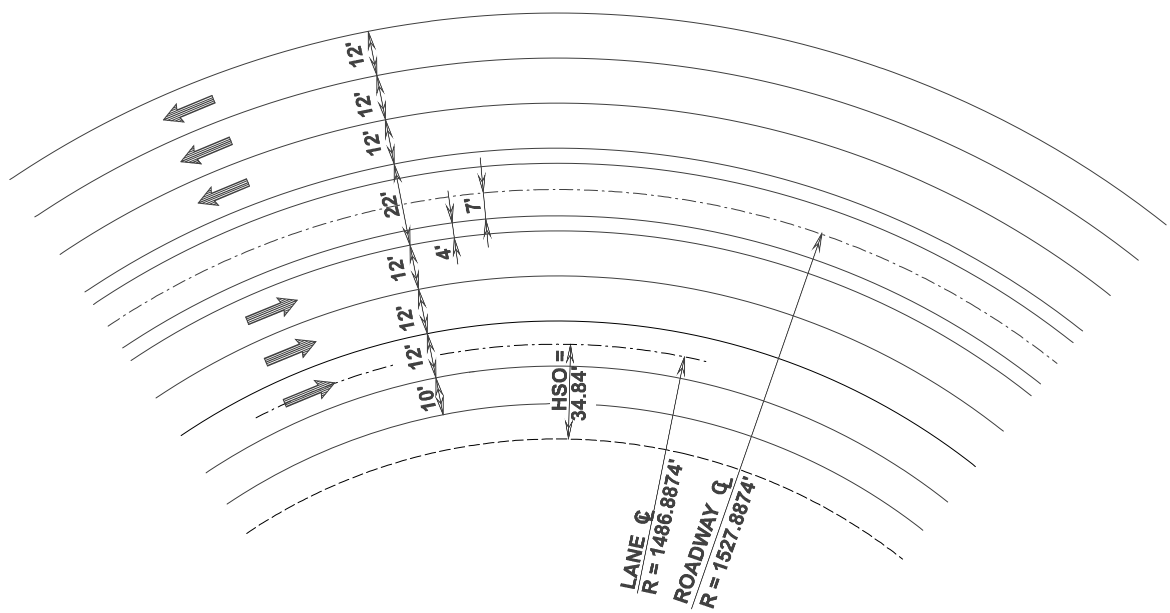
Centerline Degree of Curvature, D is 3^o 45'

If single face concrete barrier be installed along the highway without encroaching on the required horizontal sight distance, how far (at minimum) from the roadway centerline should the face of the barrier be located?

SOLUTION 19

From CERM Table 79.2 for V = 65 mph, SSD = 645 ft.

For D = 3^o 45'= 3.75 ^{o} ……. R= $\dfrac{5729.578}{3.75^\circ}$ = 1527.89 ft



- The centerline of the curve's inside lane is offset 41 ft (7 + 4+ 2(12) + (12/2)) from the roadway centerline.
- Radius of centerline of the curve's inside lane 1527.89 ft – 41 ft = 1486.89 ft

- Using CERM equation 79.45,

$$HSO = R\left(1-\cos\left(\frac{28.65S}{R}\right)\right) = 1486.8874\times\left(1-\cos\left(\frac{28.65\times 645}{1486.89}\right)\right)$$

$$HSO = 34.84\ ft$$

Required offset from centerline: 75.84 ft = (7 + 4+ 2(12) + 6 + 34.84)

PROBLEM 20

Determine the maximum safe design speed on a bicycle track with an 80-foot radius horizontal curve and 10-foot high walls located on both sides, 8 feet to the left and right of centerline. The track is on a level grade with coefficient of friction 0.25.

(A) 13 mph (B) 15 mph (C) 17 mph (D) 19 mph

SOLUTION 20

Use equation 79.44 in CERM:

$$S = \frac{R}{28.65}\left[cos^{-1}\left(\frac{R-HSO}{R}\right)\right]$$

Where:

R	=	Radius of centerline of lane (ft).
HSO	=	Distance from centerline of lane to obstruction (ft).
S	=	Stopping Sight Distance (ft)

Apply the above equation:

$$S = \frac{80}{28.65}\left[cos^{-1}\left(\frac{80-8}{80}\right)\right] = 72\ ft$$

Knowing that S = 72 ft, use the following equation to solve for V.

$$S = 1.47\ Vt + \frac{V^2}{30(f \pm G)};\quad 72 = 1.47 * 2.5\ V + \frac{V^2}{30(0.25 \pm 0)}$$

$$\frac{V^2}{7.5} + 3.67V - 72 = 0;\quad V^2 + 27.52\ V - 540 = 0$$

At this point you can solve for V exactly using the quadratic equation,

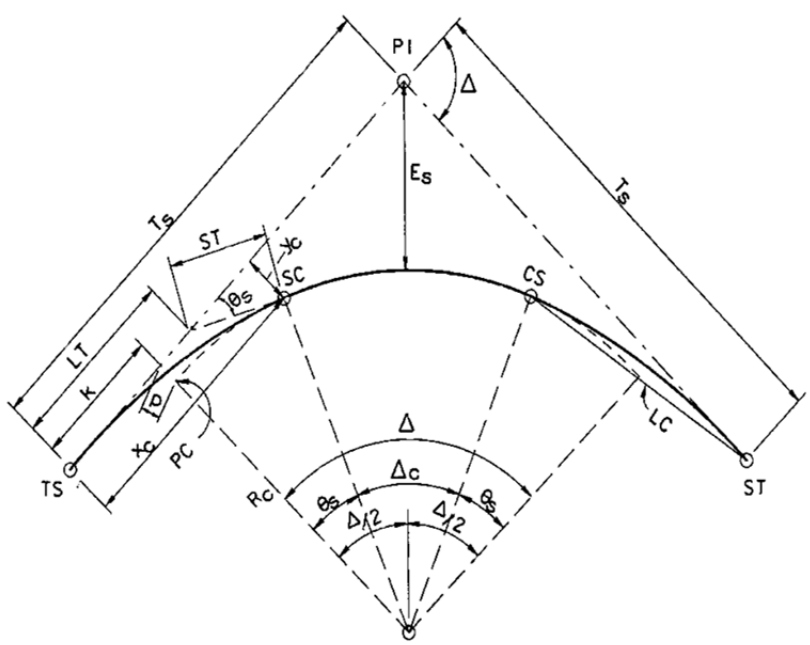
$$V = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a} = \frac{-27.52 \pm \sqrt{27.52^2 - 4(1)(-540)}}{2(1)} = \frac{-27.52 \pm 54.01}{2}$$
$$= 13.26; -40.76$$

The result: V= 13.3 mph

Therefore the correct answer is A.

3.5 Spiral Curves

A spiral is a curve of continuously changing radius. It varies from 0 degree of curvature (infinite radius) at its TS (Tangent to Spiral) to the specific degree of curvature, D_C, of the curve it connects to at the SC (Spiral to Curve). For more about spiral curves see the figure below and CERM Pages 79-18 to 79-20.



3.6 Compound Curves

See the AASHTO Green Book, 2011 edition, page 3-84

- Compound curves are advantageous in effecting desirable shapes of turning roadways for at-grade intersections and for interchange ramps

- On compound cures for open highways, it is generally accepted that the ratio of the flatter radius to the sharper radius should not exceed 1.5:1

- For compound curves at intersections where drivers accept more rapid changes in direction and speed, the radius of the flatter arc can be as much as 100 percent greater than the radius of the sharper arc, a ratio of 2:1

PROBLEM 21

The horizontal alignment of an interchange exit ramp consists of a series of three consecutive and progressively sharper circular curves that form a single compound circular curve. Proceeding in the direction of traffic, if the first curve has a radius of 2,000 ft, what is the <u>minimum</u> radius of the third curve?

SOLUTION 21

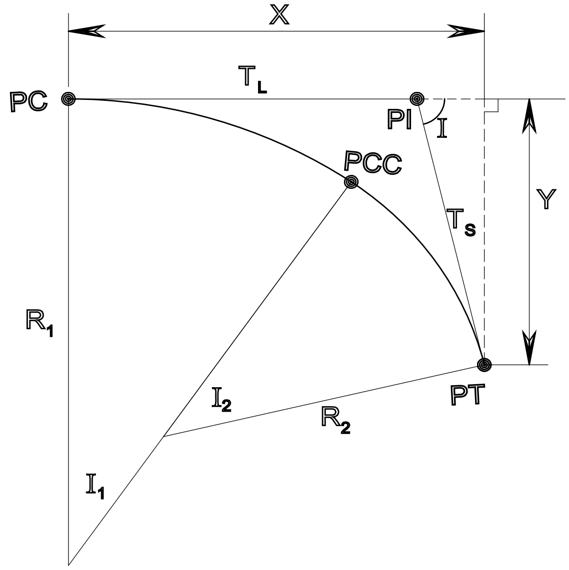
Radius of first curve = 2,000 ft

Minimum radius of the second curve (sharper than the first curve) = Half of the radius of the first curve = 1,000 ft

Minimum radius of the third curve (sharper than the second curve) = Half of the radius of the second curve = 500 ft

3.6.1 Two-centered Compound Curve

Components of a two-centered compound curve are shown in the following figure:



PI = Point of Intersection of back tangent and forward tangent.
PC = Point of Curvature—point of change from back tangent to circular curve.
PT = Point of Tangency—point of change from circular curve to forward tangent.
PCC = Point of Compound Curvature.
T_L = Long Tangent of the compound curve.
T_S = Short Tangent of the compound curve.
I = Total intersection angle of the compound curve.
X = Distance from PC to PT in the direction of the backward tangent.
Y = Perpendicular distance from the backward tangent to the PT.
I_1 = Intersection angle of the flatter curve (decimal degrees).
I_2 = Intersection angle of the sharper curve (decimal degrees).
R_1 = Radius of the flatter curve.
R_2 = Radius of the sharper curve

$$I = I_1 + I_2$$

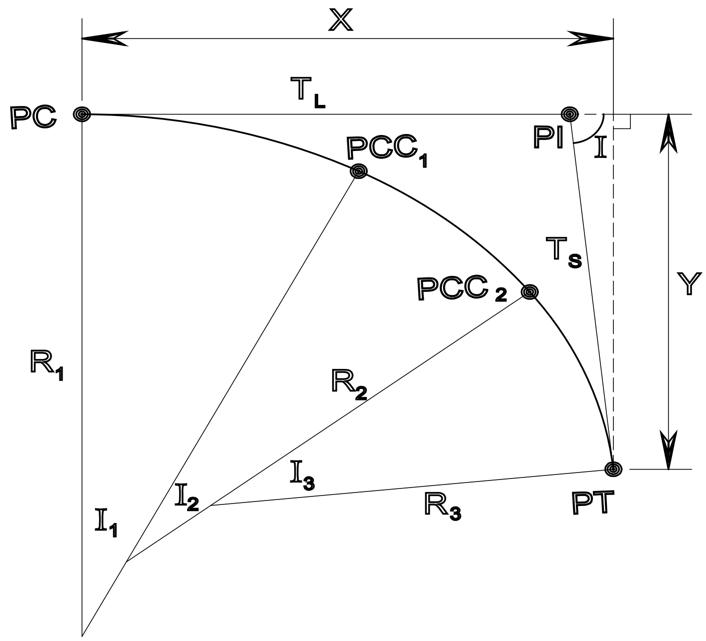
$$X = R_2 \times \sin I + (R_1 - R_2) \times \sin I_1 \quad ; \quad Y = R_1 - R_2 \times \cos I - (R_1 - R_2) \times \cos I_1$$

$$T_L = \frac{R_2 - R_1 \times \cos I + (R_1 - R_2) \times \cos I_2}{\sin I}$$

$$T_S = \frac{R_1 - R_2 \times \cos I - (R_1 - R_2) \times \cos I_1}{\sin I}$$

3.6.2 Three-centered Compound Curve

Components of a three-centered compound curve are shown in the following figure:



Definitions

PI = Point of Intersection of back tangent and forward tangent.
PC = Point of Curvature—point of change from back tangent to circular curve.
PT = Point of Tangency—point of change from circular curve to forward tangent.
PCC = Point of Compound Curvature.
T_L = Long Tangent of the compound curve.
T_S = Short Tangent of the compound curve.
I = Total intersection angle of the compound curve.
X = Distance from PC to PT in the direction of the backward tangent.
Y = Perpendicular distance from the backward tangent to the PT.
I_1 = Intersection angle of the flatter curve (decimal degrees).
I_2 = Intersection angle of the middle curve (decimal degrees).
I_3 = Intersection angle of the sharper curve (decimal degrees).
R_1 = Radius of the flatter curve.
R_2 = Radius of the middle curve
R_3 = Radius of the sharper curve

Formulas

$I = I_1 + I_2 + I_3$

$X = (R_1 - R_2) \times \sin I_1 + (R_2 - R_3) \times \sin(I_1 + I_2) + R_3 \times \sin I$

$Y = R_1 - R_3 \times \cos I - (R_1 - R_2) \times \cos I_1 - (R_2 - R_3) \times \cos(I_1 + I_2)$

$$T_L = \frac{R_3 - R_1 \times \cos I + (R_1 - R_2) \times \cos(I_2 + I_3) + (R_2 - R_3) \times \cos I_3}{\sin I}$$

$$T_S = \frac{R_1 - R_3 \times \cos I - (R_1 - R_2) \times \cos I_1 - (R_2 - R_3) \times \cos(I_1 + I_2)}{\sin I}$$

PROBLEM 22

Given the following for a three-centered compound curve:

R_1=500 ft;
R_2=350 ft;
R_3=200 ft;
$I_1=30^0$;
$I_2=35^0$; and
$I_3=40^0$

Find the following:

a) Total intersection angle
b) Distance from point of curvature to point of tangency
c) Perpendicular distance from the backward tangent to the point of tangency
d) Long tangent
e) Short tangent

SOLUTION 22

a) $I = 105^0$
b) X = 404.13 ft
c) Y = 358.47 ft
d) T_L = 500.18 ft
e) T_S = 371.11 ft

4. Vertical Curves

NOTE: Some portions of this section are reviewed under Geometrics section of the regular classes also. Necessary fundamental topics are repeated here to have a quick overview before going into depth problems.

- Vertical curves are used to change the grade of a highway.
- Most vertical curves take the shape of an equal-tangent parabola. Such curves are symmetrical about the vertex.
- Two types of vertical curves – Crest and Sag

See CERM figure 79.10 and equations 79.46 to 79.49 on pages 79-11 and 79-12.

4.1 Vertical Curve Elevations

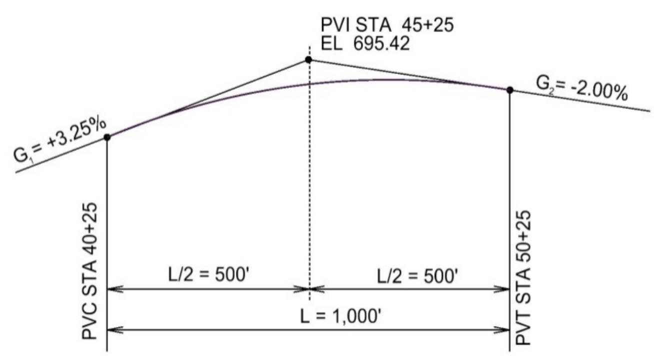
PROBLEM 23

A +3.25% grade intersects a -2.00% grade at Sta. 45+25 and elevation 695.42 ft. A 1000 ft vertical curve connects the two grades. Determine:

a) the station of turning point
b) the elevation of turning point

SOLUTION 23

a) Using CERM Eqn. 79.48, $x = -G_1/R$



Using CERM eqn. 79.46,

$$R = \frac{G_2 - G_1}{L_{Sta}} = \frac{-2.00 - 3.25}{10} = -0.525$$

$x = -G_1/R = -3.25/-0.525 = 6.1905$ Sta.

Highest Point Location:

Sta. = BVC Sta. + x = 4025 ft + 619.05 ft = 4644.05 ft = **Sta 46+44.05**

b) Using CERM equation 79.47, *Elev.* $= (R/2)x^2 + G_1(x) +$ *BVC Elev.*

Elevation at BVC = 695.42 - 5(3.25) = 679.17 ft

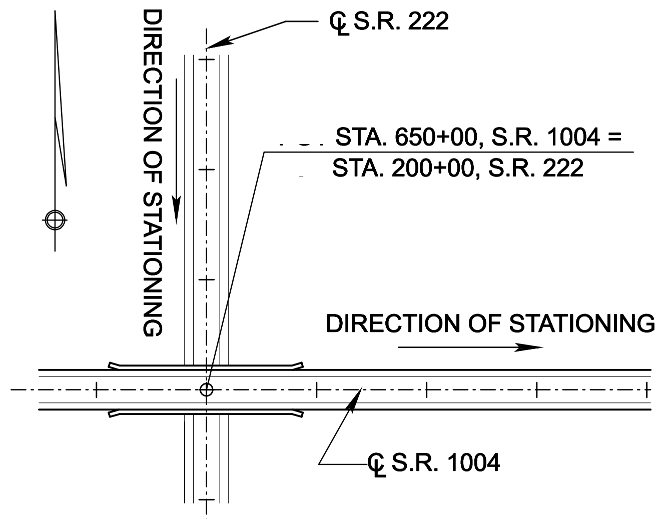
Elev. = $(R/2)x^2 + G_1(x) +$ BVC Elev. = 689.23 ft

Where $x = 6.1905$, $R = -0.525$, $G_1 = 3.25$, BVC Elev = 679.17

PROBLEM 24

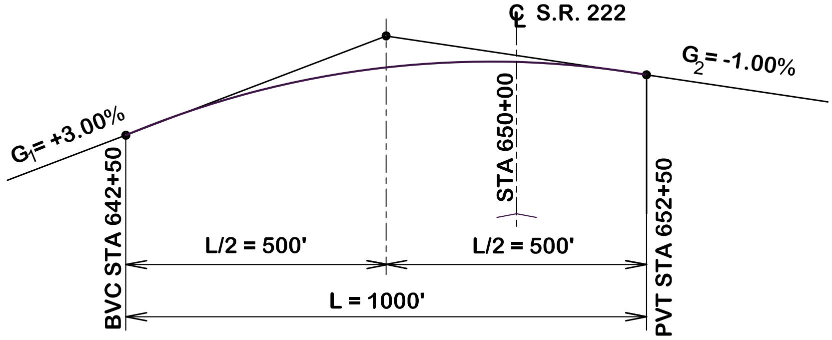
State Route (S.R.) 1004 crosses over S.R. 222 as shown in the figure. The overpass structure depth is 5.25' and the vertical alignment of each street is provided in the table. Determine the vertical clearance between the S.R. 222 profile grade and the bottom of structure at the intersection point.

	S.R. 1004	**S.R. 222**
PVI Station	647+50	199+00
PVI Elev	1200.00	1172.50
L =	1000 ft	600 ft
G_1 =	+3.0 %	-3.0%
G_2 =	-1.0 %	+1.5%



SOLUTION 24

(a) Draw a rough sketch of the S.R. 1004 profile.



PROFILE
S.R. 1004
NOT TO SCALE

(b) Find the profile grade elevation at Sta. 650+00 on S.R. 1004.

$$R = \frac{G_2 - G_1}{L} = \frac{-1-(+3)}{10} = \frac{-4}{10} = -0.4;$$

$$BVC\ Elev = 1200.00 - 5(3) = 1185.00\ \textbf{ft};$$

$$BVC\ Sta = 64750 - \frac{1000}{2} = 64250 = Sta\ 642+50;$$

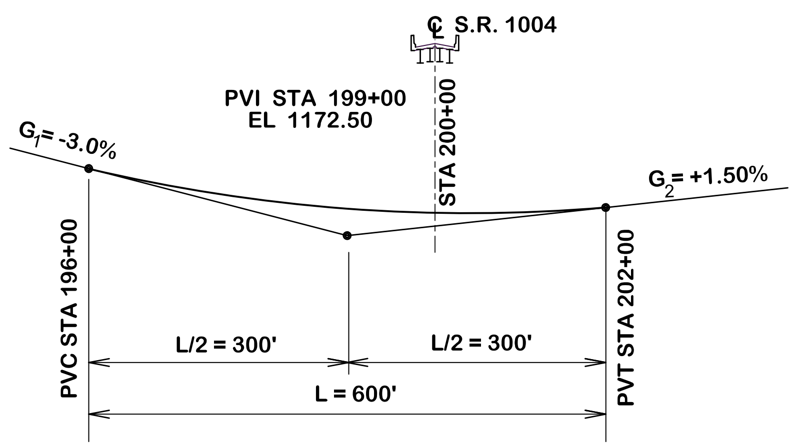
$$x = 65000 - 64250 = 750' = 7.5\ sta$$

$$Elev\ at\ Sta.\ 650+00:$$

$$\frac{R}{2}x^2 + G_1(x) + BVC\ Elev = \frac{-0.4}{2}(7.5)^2 + 3.0(7.5) + 1185.00$$

$$= -11.25 + 22.50 + 1185 = \underline{\underline{1196.25}}\ \textbf{ft}$$

(c) Draw a rough sketch of the S.R. 222 profile.



PROFILE
S.R. 222
NOT TO SCALE

(d) Find the profile grade elevation at Sta. 200+00 on S.R. 222.

$$R = \frac{G_2 - G_1}{L} = \frac{1.5 - (-3)}{6} = \frac{4.5}{6} = 0.75;$$

$$BVC\ Elev = 1172.50 + 3(3) = 1181.50\ \mathbf{ft};$$

$$BVC\ Sta = 19900 - \frac{600}{2} = 19600 = Sta\ 196 + 00;$$

$$x = 20000 - 19600 = 400' = 4.0\ sta$$

$$Elev\ at\ Sta.\ 200 + 00:$$

$$\frac{R}{2}x^2 + G_1(x) + BVC\ Elev = \frac{0.75}{2}(4.0)^2 - 3.0(4.0) + 1181.50$$

$$= 6.00 - 12.00 + 1181.50 = \underline{\underline{1175.50}}\ \mathbf{ft}$$

(e) The vertical clearance is the difference in profile grade elevations minus the structure depth.

$$1196.25 - 1175.50 - 5.25' = \underline{\underline{15.50\ ft.}}$$

4.2 Vertical Curve Design

- Using AASHTO Guidelines
- Minimum vertical curve length is computed
- Based on sight distance criteria
- Use CERM Table 79.4
- If the length should be based on K-value method, use the following guidelines:

  L = KA (CERM equation 79.57)

  Where $A = |G_2 - G_1|$

  K = Factor that can be found using the following references based on the type of the curve

Units	Stopping Sight Distance (Crest Curves)	Passing Sight Distance (Crest Curves)	Headlight/Stopping Sight Distance (Sag Curves)
SI Units	CERM Fig 79.12	AASHTO Table 3-35	CERM Fig 79.14
US Units	CERM Fig 79.13	AASHTO Table 3-35	CERM Fig 79.15

Table 3-35. Design Controls for Crest Vertical Curves Based on Passing Sight Distance

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

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

SOURCE: A Policy on Geometric Design of Highways and Streets by AASHTO

PROBLEM 25

Given a two-lane roadway with $G_1 = +2.0\%$, $G_2 = -2.5\%$, and a design speed of 60 mph, determine:

1. The minimum length of vertical curve for stopping sight distance using K-value?
2. The minimum length of vertical curve for passing sight distance using K-value?

SOLUTION 25

Part 1-

- Determine the algebraic difference in grades, A.

$$|A| = |G_2 - G_1| = |-2.5 - 2| = |-4.5| = 4.5$$

- L_{min} for stopping sight distance, first verify that stopping sight distance is appropriate for the type of vertical curve indicated. Stopping sight distance applies to crest curves only. It often helps to draw a rough sketch. Since this is a crest curve, proceed to CERM Figure 79.13.

- Using CERM Figure 79.13:

$$L_{\min} = K \times |A| = 151 \times 4.5 = 679.5 \quad \approx \underline{\underline{680 \quad feet}}$$

Part 2 -

- L_{min} for passing sight distance, first verify that passing sight distance is appropriate for the typical section indicated. Passing sight distance applies to two-lane roadways only. Since this is a two-lane roadway, proceed to the table provided above.

- From Table 3-35 shown on the previous page, reading across the row marked, 60 mph, find K=357, and calculate L_{min}:

$$L_{\min} = K \times |A| = 357 \times 4.5 = \underline{\underline{1606.50 \quad feet}}$$

PROBLEM 26
Given a two-lane highway with a 1,400 ft vertical curve with G_1=+0.50%; G_2= -0.25%; PVI 85+00; PVI elevation = 457.59 feet; and a Design Speed of 65 mph. Determine the actual Passing Sight Distance provided on the curve.

SOLUTION 26

From CERM Table 79.4,

Assume S < L;

$$\text{Therefore } S = \sqrt{\frac{2800L}{A}} = \sqrt{\frac{2800(1400)}{0.75}} = \sqrt{\frac{2800(1400)}{0.75}} = \underline{\underline{2286.19}}$$

Since S = 2286.19 feet > L = 1400 feet, the initial assumption, (S < L), is No Good.

Therefore, assume the opposite case is true: S > L

$$L = 2S - \frac{2800}{A} \longrightarrow S = \frac{1}{2}\left[L + \frac{2800}{A}\right]$$

Solving for S

$$S = \frac{1}{2}\left[1400 + \frac{2800}{0.75}\right] = \frac{1}{2}\left[5133.33\right] = \underline{\underline{2566.67}}$$

S = 2566.67 feet > 1400 feet. This is consistent with the opposite assumption: S > L.

Therefore $\mathbf{S_{actual} = 2566.67}$ **feet**

PROBLEM 27

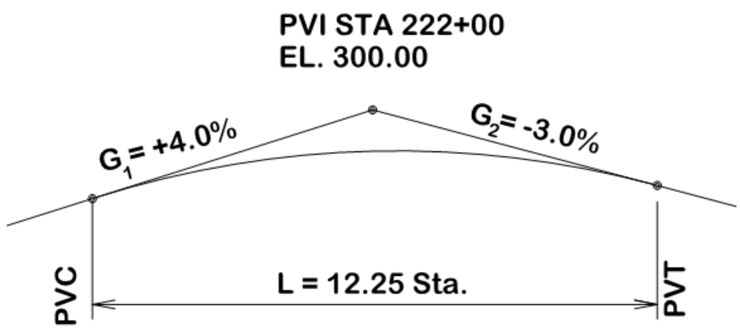
A +4.0% grade intersects a -3.0% percent grade at PVI Sta. 222+00 and Elev. 300.00 on a two-lane highway with a design speed of 45 mph. What is the turning point elevation for the curve that is designed to meet passing sight distance using K-value method?

SOLUTION 27

STEP 1: Compute the length of the curve based on Passing Sight Distance using K-value method?

Using AASHTO Table 3-35 included in this chapter in the previous pages, for V=45 mph …. K = 175;

L = K(A) = 175(7) = 1225'= 12.25 sta

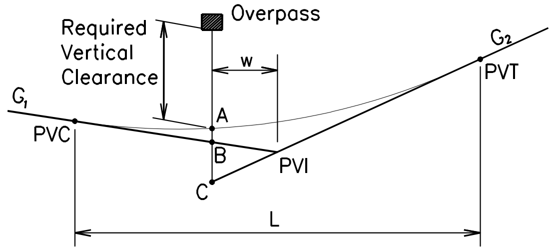


STEP 2: Calculate <u>the highpoint elevation</u>

4.3 Fixed Point on the Vertical Curve

Given the elevation of a fixed point on vertical curve, length of the curve can be determined using the following technique (See CERM Page 79-13):

4.3.1 Fixed Point is located anywhere along the curve



Procedure:
Step 1. Draw a rough sketch identify points A, B, and C.
Step 2. Calculate the elevations of points A, B, and C.
Step 3. Calculate the constant z, (no physical significance).

(NOTE: In CERM, ‘z’ is shown as ‘s’ and ‘w is shown as ‘d’)

$$z = \sqrt{\frac{ElevA - ElevC}{ElevA - ElevB}}$$

Step 4. Solve the following equation for L.

$$L = \frac{2w(z+1)}{z-1}$$

‘w’ is in stations and therefore, L value will be in stations

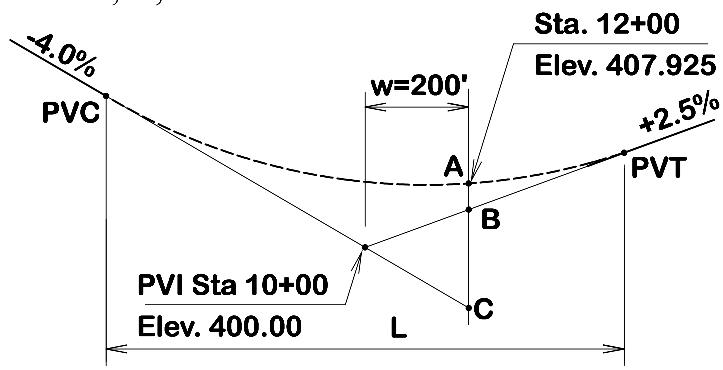
PROBLEM 28

Calculate the length of a sag vertical curve that passes through a point located at elevation 407.925 and Sta. 12+00.

Given: G_1 = - 4.0%; G_2= + 2.5%; PVI_{Sta} =10+00.00; $Elev_{pvi}$= 400.00

SOLUTION 28

Step 1. Draw a rough sketch and identify points A, B, and C.



Step 2. Determine the elevations of points A, B, and C.

Elev A; given: 407.925

Elev B; Point B is located on G_2,

$$ElevB = Elev(pvi) + w \times G_2$$
$$ElevB = 400.00 + 2 \times 2.5 = \underline{\underline{405.00}}$$

Elev C; Point C is located on G_1,

$$ElevC = Elev(pvi) + w \times G_1$$
$$ElevC = 400.00 + 2 \times (-4.0) = 400.00 - 8.0 = \underline{\underline{392.00}}$$

Step 3. Calculate the constant z, (no physical significance).

$$z = \sqrt{\frac{ElevA - ElevC}{ElevA - ElevB}} = \sqrt{\frac{407.925 - 392.00}{407.925 - 405.00}} = \sqrt{\frac{15.925}{2.925}} = \sqrt{5.4444} = 2.333$$

Step 4. Solve the following equation for L.

$$L = \frac{2w(z+1)}{z-1} = \frac{2(2)(2.333+1)}{(2.333-1)} = 10.00 \text{ sta} = 1{,}000 \text{ ft}$$

4.3.2 Fixed Point is located at turning point along the curve

See CERM equation 79.52 on page 79-14

$$L = \frac{2(G_2 - G_1)(Elev_{PVI} - Elev_{TP})}{G_1 G_2}$$

5. Intersection Design

5.1 Width of Turning Roadway at Intersection

PROBLEM 29
A proposed intersection ramp will carry predominantly P vehicles but some consideration must also be given to occasional SU trucks. The pavement width is to be designed for one-lane, one-way operation with provisions for passing a stalled vehicle so traffic flow can be maintained at reduced speeds. The minimum radius on the inner edge of pavement is to be 75 feet and vertical curb is to be place on both sides of the pavement. Based on this information determine the required pavement width.

SOLUTION 29
See the AASHTO Green Book (2011), page 3-97 for the discussion of turning roadway operational classifications. As the problem uses US Customary units refer to the right half of Table 3-29.

The situation described above includes: Case II operations and Design Traffic Condition A. Enter Table 3-29 using these values. With a 75'foot radius on the inner edge of pavement, the required basic pavement width required is: 19'.

Because vertical curb is to be placed on two sides, add 1' to the basic pavement width. Therefore, **<u>the required total width is 19' + 1' = 20'.</u>**

5.2 Intersection Sight Distance

Reference – AASHTO Green Book (2011) pages 9-28 thru 9-41

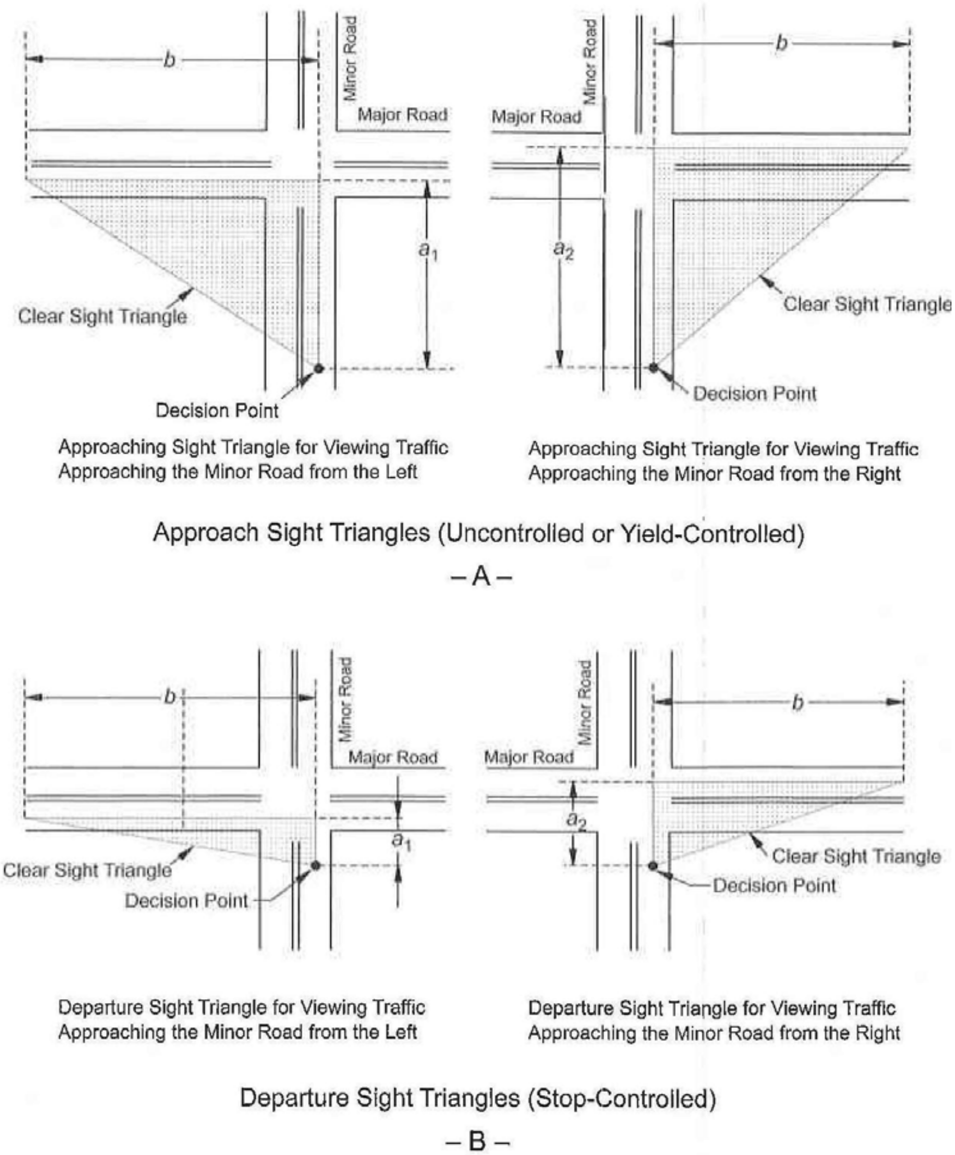


Figure 9-15. Intersection Sight Triangles

Recommended dimensions of the sight triangles vary with the type of traffic control used at an intersection. We will do examples to calculate ‘b’ value for Stop control which is the most important for the exam. To find out ‘a_1’ and ‘a_2’ values, please read the underlined text on page 9-36 of AASHTO Green Book (2011) in the attached Reference Material section.

Following are different types of procedures that are further defined in AASHTO Green Book (2011):

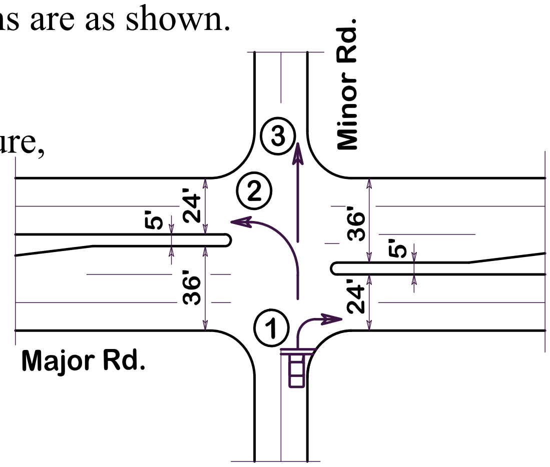
Case A – Intersections with no control
Case B – Intersections with stop control on the minor road
- Case B1 – Left turn from the minor road
- Case B2 – Right turn from the minor road
- Case B3 – Crossing maneuver from the minor road

Case C – Intersection with yield control on the minor road
- Case C1 – Crossing from the minor road
- Case C2 – Left or right turn from the minor road

Case D – Intersections with traffic signal control
Case E – Intersections with all-way stop control
Case F – Left turns from the major road

PROBLEM 30

The figure at right shows a design vehicle stopped on a minor road at a major road intersection. The design speed of the major road is 60 mph. The minor road approach grade is +3.4 percent. The lane and median widths are as shown.



For each of the three maneuvers shown in the figure, determine the design intersection sight distance assuming the design vehicle is a:

- Passenger car
- Single unit truck
- Combination truck

SOLUTION 30

See AASHTO Green Book (2011) Intersection Control, Cases B1, B2, and B3; pages 9-28 thru 9-41. For all three maneuvers, use the following equation:
$b = ISD = 1.47V_{major}\, t_g$. With $V_{major} = 60$ mph, the equation becomes $ISD = 88.2t_g$. Determine t_g by adjusting for additional lanes and steep approach grades per Tables 9-5 and 9-7.

NOTE: "The median width should be considered in determining the number of lanes to be crossed. The median width should be converted to equivalent lanes. For example, a 24-ft median should be considered as two additional lanes to be crossed in applying the multilane highway adjustments for time gaps…"

SOLUTION TABLE							
			Components of time gap, t_g			Total ISD (ft)	Reference
Turn	Design Vehicle	$1.47V_g$	Basic t_g	Adjustment for additional lanes crossed	Adjustment for approach grade		
1 Right	Passenger Car	88.2[	6.5 +	0 +	0.1(3.4)] =	603	AASHTO pg 664
	Single Unit Truck					780	
	Combination Truck					956	
2 Left	Passenger Car	88.2[	7.5 +	0.5(2.42) +	0.2(3.4)] =	828	AASHTO pg 660
	Single Unit Truck						
	Combination Truck						
3 Thru	Passenger Car	88.2[	6.5 +	0.5(2.42) +	0.1(3.4)] =	754	AASHTO pg 664
	Single Unit Truck						
	Combination Truck						

6. Interchange Design

6.1 Minimum Length Between Ramp Termini

PROBLEM 31

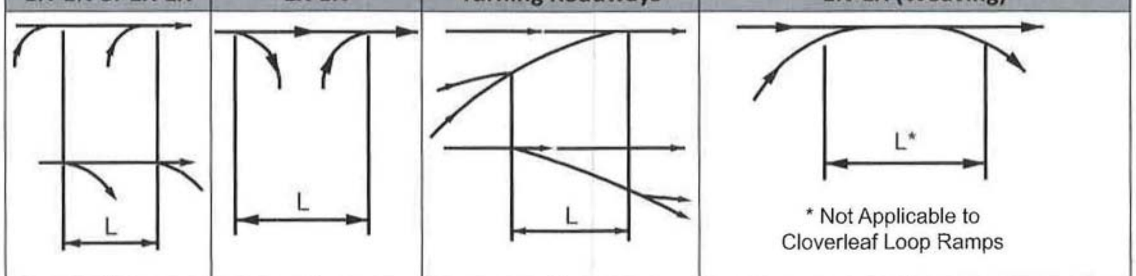
(a) What is the recommended minimum length, L, between successive entrance ramp terminals on an FDR?

(b) What is the recommended minimum length, L, between successive entrance ramp and exit service to service interchanges on the full freeway?

(c) What is the recommended minimum length, L, between successive entrance ramp terminals of service interchanges on the turning roadways?

SOLUTION 31

Referring to the Figure below, (a) L = 800 ft; (b) L = 1600 ft; (c) L = 600 ft



EN-EN or EX-EX		EX-EN		Turning Roadways		EN-EX (Weaving)			
Full Freeway	CDR or FDR	Full Freeway	CDR or FDR	System Interchange	Service Interchange	System to Service Interchange		Service to Service Interchange	
						Full Freeway	CDR or FDR	Full Freeway	CDR or FDR
Minimum Lengths Measured between Successive Ramp Terminals									
300 m (1000 ft)	240 m (800 ft)	150 m (500 ft)	120 m (400 ft)	240 m (800 ft)	180 m (600 ft)	600 m (2000 ft)	480 m (1600 ft)	480 m (1600 ft)	300 m (1000 ft)

Notes: FDR—Freeway distributor road EN—Entrance
CDR—Collector distributor road EX—Exit

Figure 10-68. Recommended Minimum Ramp Terminal Spacing

Source: AASHTO Green Book (2011)

6.2 Minimum Acceleration Length for Entrance Terminal

PROBLEM 32

A new interchange will be located on an east-west highway with a design speed of 65 mph. The eastbound entrance ramp will have a +1.9% grade and an entrance curve with a design speed of 50 mph. The westbound entrance ramp will have a -2.00% grade and an entrance curve with a design speed of 35 mph. Both ramps will be single-lane, parallel type ramps.

(a) What is the minimum acceleration length, L, on the eastbound entrance ramp?

(b) What is the initial running speed, V_a', on the eastbound entrance ramp entrance curve?

(c) What is the minimum acceleration length, L, on the westbound entrance ramp?

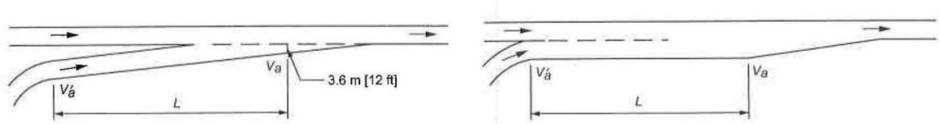
(d) What is the initial running speed, V_a', on the westbound entrance ramp entrance curve?

SOLUTION 32 Referring to the table below:

(a) L = 370 ft; (b) V_a' = 44 mph; (c) L = 1000 ft; (d) V_a' = 30 mph

U.S. Customary										
		Acceleration Length, L (ft) for Entrance Curve Design Speed (mph)								
Highway		Stop Condition	15	20	25	30	35	40	45	50
		and Initial Speed, V'_a (mph)								
Design Speed, V (mph)	Speed Reached, V_a (mph)	0	14	18	22	26	30	36	40	44
30	23	180	140	—	—	—	—	—	—	—
35	27	280	220	160	—	—	—	—	—	—
40	31	360	300	270	210	120	—	—	—	—
45	35	560	490	440	380	280	160	—	—	—
50	39	720	660	610	550	450	350	130	—	—
55	43	960	900	810	780	670	550	320	150	—
60	47	1200	1140	1100	1020	910	800	550	420	180
65	50	1410	1350	1310	1220	1120	1000	770	600	370
70	53	1620	1560	1520	1420	1350	1230	1000	820	580
75	55	1790	1730	1630	1580	1510	1420	1160	1040	780

Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1,300 ft.



Taper Type Parallel Type

Table 10-3. Minimum Acceleration Lengths for Entrance Terminals with Flat Grades of Two Percent or Less.

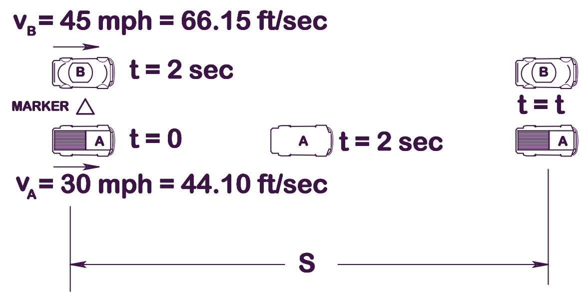
Source: AASHTO Green Book (2011), page 10-110

7. Traffic Characteristics

See CERM page 71-4 for relationships between Speed, Distance, Time, and Acceleration

PROBLEM 33

Refer to the figure at right.



(a) At t = 0, vehicle A is traveling at a speed of 30 mph and passes a road marker on a straight section of a highway. Vehicle B, traveling with a speed of 45 mph, passes the marker 2 seconds later. Find the time when vehicle B overtakes vehicle A, and the corresponding distance, s, from the road marker.

(b) Do the same analysis as in part (a) assuming vehicle B starts to accelerate at the rate of 2 ft/s^2 at the instant that it passes the road marker.

SOLUTION 33

(a) The figure above shows a sketch of the events. S is the distance from the marker to where B overtakes A at time t. From the figure, $s = s_A = s_B$

Using $s = vt$, $44.10t = 66.15(t-2)$, $44.10t = 66.15t - 132.30$; $t = 6.0$ sec

$s = s_A = s_B = 44.10(6.0) = 264.60$ ft.

(b) Using the lower figure, $s = s_A = s_B$

Using $s_A = v_A t$ and $s_B = v_{0B}t + ½\, a_B\, t^2$

$44.10\, t = 66.15(t – 2) + ½\, (2)(t – 2)^2$

Simplifying: $t^2 + 18.05t – 128.3 = 0$

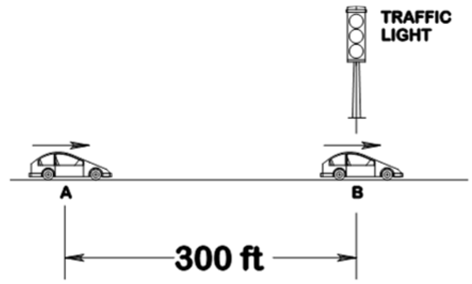
Use the quadratic formula to solve for t.

$$t = 5.46 \text{ sec};$$

$$s = v_A t = 44.10(5.46) = 240.79\ ft$$

PROBLEM 34

Refer to the figure at right. Vehicle B is stopped at a traffic signal, as shown in the figure at right. At the instant the light turns green, vehicle B starts to accelerate at 3 ft/sec^2. At the same time vehicle A is 300 ft behind vehicle B, traveling at a speed of 30 mi/h.



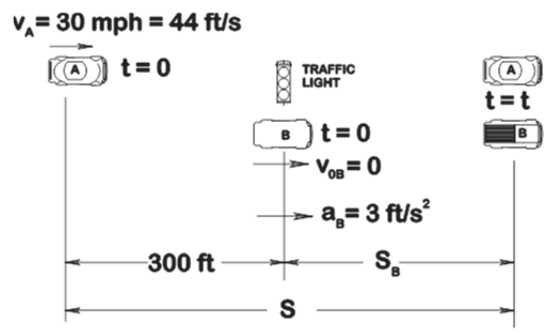
(a) At what distance past the traffic signal will vehicle A overtake vehicle B assuming Vehicle A is travelling faster than Vehicle B?

(b) At what distance past the light will vehicle B overtake vehicle A assuming Vehicle B is travelling faster than Vehicle A?

(c) At what acceleration of vehicle B, vehicle A can only join Vehicle B but can't overtake?

SOLUTION 34

(a) The figure at right shows a sketch of the events. The displacement coordinate s is measured from the position of vehicle A at t = 0. The position when both vehicles are side by side is s.



$s = s_A = 300 + s_B \quad v_A t = 300 + ½\, a_B\, t^2$

$44t = 300 + ½\, (3)\, t^2$

$1.5t^2 – 44t + 300 = 0$

T=10.8 sec, 18.6 sec

Using t = 10.8sec,

$v_A = 44$ ft/s = constant

$v_B = a_B t = 3(10.8) = 32.4$ ft/sec

Thus $v_A > v_B$, and vehicle A overtakes vehicle B.

Using t = 18.6sec,

$v_A = 44$ ft/sec = constant

$v_B = a_B t = 3(18.6) = 55.8$ ft/sec

Since $v_A < v_B$, vehicle A can't overtake vehicle B. (This is not our scenario)

Therefore, t =10.8 sec for our scenario

The distance past the signal is: $s_B = ½\, a_B\, t^2 = ½\, (3)\, 10.8^2 = 175$ ft

(b) Assume vehicle B continues to accelerate at 3 ft/s^2. At t = 18.6 sec,

$v_A = 44$ ft/sec = constant $v_B = a_B t = 3(18.6) = 55.8$ ft/sec

Thus $v_B > v_A$, and vehicle B overtakes vehicle A.

The distance past the signal is given by: $s_B = ½\, a_B\, t^2 = ½\,(3)\; 18.6^2 = 519$ ft

(c) When they both meet,

$s_A = 300 + s_B$ $44t = 300 + ½\, a_B\, t^2$ (1)

After they meet, both travelling at the same speed,

$v_A = v_B$ $44 = a_B t$ (2)

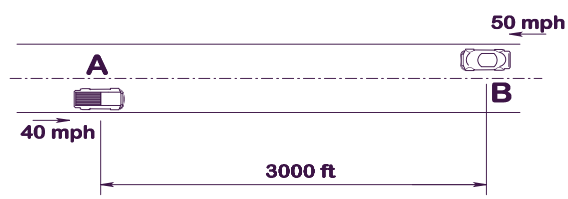
Using Eq. (2) in Eq. (1), $44t = 300 + ½\, t(44)$ $22t = 300 = 13.6$ sec

Using Eq. (2), $44 = a_B\,(13.6)$ $a_B = 3.24$ ft/sec^2

If $a_B = 3.24$ ft/sec^2, vehicle A can only join vehicle B, but can't overtake. After that point, they both travel at the same speed.

PROBLEM 35

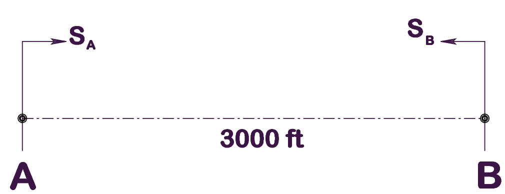
a) Two vehicles approach each other in opposite lanes of a straight horizontal roadway as shown below. At time t=0 the vehicles have the speeds and positions shown in the figure. Find the time and position at which the vehicles meet if both continue to move with constant speed.



b) Do the same as in part (a) if, at t=0, vehicle A continues to move with constant speed and vehicle B starts to accelerate at 5 ft/sec^2.

SOLUTION 35

a) The displacement coordinates of the vehicles are shown in the figure below.



The initial velocities are:

$$v_A = 40 \text{ mi/hr}\left(\frac{5{,}280 \ ft/mi}{3{,}600 \ \text{sec}/hr}\right) = 58.67 \text{ ft/sec}$$

$$v_B = 50 \text{ mi/hr}\left(\frac{5{,}280 \ ft/mi}{3{,}600 \ \text{sec}/hr}\right) = 73.33 \text{ ft/sec}$$

The time t at which the vehicles meet is found from

$$s = v\,t$$

$$s_A + s_B = 3000$$

$$58.67t + 73.33t = 3000;$$

$$t = 22.73 \text{ sec}$$

The positions of each vehicle, at the time they meet, are given by:

$s = v\,t$ $\quad s_A = 58.67(22.73) = 1333.57$ ft

$s_B = 73.33(22.73) = 1666.79$ ft

b) Vehicle A moves with constant speed and vehicle B accelerates at 5 ft/sec^2. The time at which the vehicles meet is found from:

$$s_A + s_B = 3000 \ ft \qquad (v_A t) + (v_B t + 1/2 \ a_B t^2) = 3000 \ ft$$

$$58.67t + 73.33t + \frac{1}{2} \times 5t^2 = 3000 \ ft$$

$$t = 17.15 \text{ sec}$$

8. Traffic Safety

8.1 Accident Analysis

Reference: See CERM Pages 75-7 and 75-8

Crash rates are normally considered better indicators of risk than crash frequencies.

- Crash rates for intersections are normally expressed in terms of crashes per million entering vehicles (MEV), using the following equation (See CERM equation 75.26 on page 75-7):

$$R_{int} = \frac{A * 10^6}{365 * T * V}$$

Where: R_{int} = crash rate for the intersection
A = number of reported crashes
T = time period of the analysis (years)
V = average daily traffic (ADT) volumes entering the intersection (veh/day)

- Crash rates for **roadway segments** are normally expressed in terms of crashes per 100 million vehicle-miles (100 MVM), using the following equation (See CERM equation 75.27 on page 75-8):

$$R_{sec} = \frac{A * 10^8}{365 * T * V * L}$$

Where: R_{sec} = crash rate for the roadway section
A = number of reported crashes
T = time period of the analysis (years)
V = average daily traffic (ADT) volumes (veh/day)
L = length of the segment (miles)

PROBLEM 36

An intersection has a total entering traffic volume of 42,000 vehicles per day. During the past three years, there have been a total of 35 reported intersection-related crashes. What is the crash rate for this intersection? (NOTE: Instead of giving the total entering traffic volume, the problem might state entering traffic volume by approach. In that case, volumes from all approaches to be added to get the total entering traffic volume.)

SOLUTION 36

$$R_{int} = \frac{35 \times 10^6}{365 \times 3 \times 42{,}000} = 0.76 \text{ crashes per MEV}$$

PROBLEM 37

A five-mile long section of two-lane road has an ADT of 8,000. There have been six crashes on this section of road during the past two years. What is the crash rate?

SOLUTION 37

$$R_{SEC} = \frac{6 \times 10^8}{365 \times 2 \times 8{,}000 \times 5} = 20.5 \text{ crashes per 100 MVM}$$

PROBLEM 38

At High Street and Main Street intersection there were 3 fatalities in 28 total accidents in the last three years. What is the severity index?

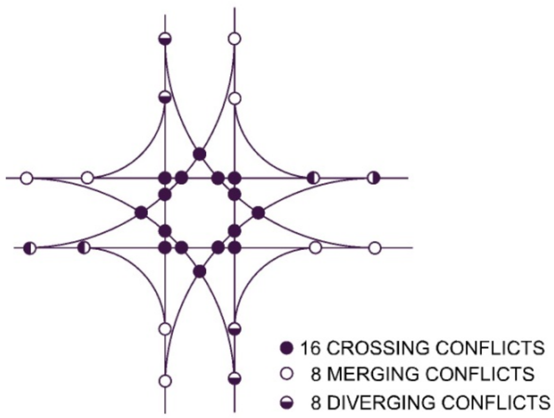
SOLUTION 38

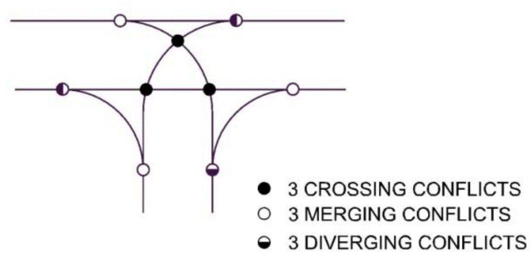
$$SI = \frac{3}{28} = 0.107 \text{ deaths/accident}$$

8.2 Intersection Conflict Points

Conflict points at intersections occur where vehicular movements cross each other, merge, and diverge. Each conflict point is a location for potential delays and a hazard.

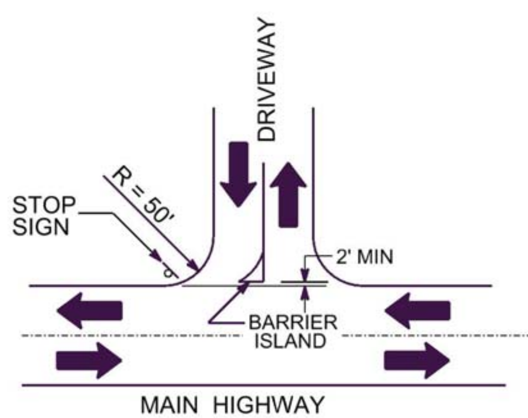
A typical four-leg intersection would have a total of 32 conflict points and a typical three-leg intersection would have a total of 9 conflict points as shown below:





PROBLEM 39

The intersection shown below includes a driveway from a shopping complex accessing a heavy traffic highway. Recent crash data for this intersection indicates a growing trend caused by the left turn movements coming out of the driveway. The city is considering eliminating the left turns coming out of the driveway by constructing a barrier island as shown in the figure. If the barrier island is constructed,



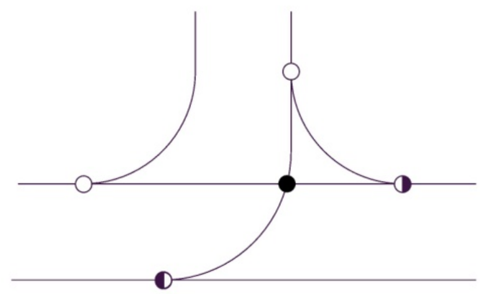
(a) how many conflict points would be eliminated?

(b) how many of the remaining conflict points would be diverging conflicts?

SOLUTION 39

A typical three-leg intersection would have a total of 9 conflict points. As shown in the following figure, there will be a total of 5 conflict points if the barrier island is placed.

(a) 4 conflict points would be eliminated with the proposed plan
(b) 2 diverging conflicts – one for each direction of the Main Highway



9. Modern Roundabouts

See AASHTO Green Book (2011) pages 9-167 thru 9-176.

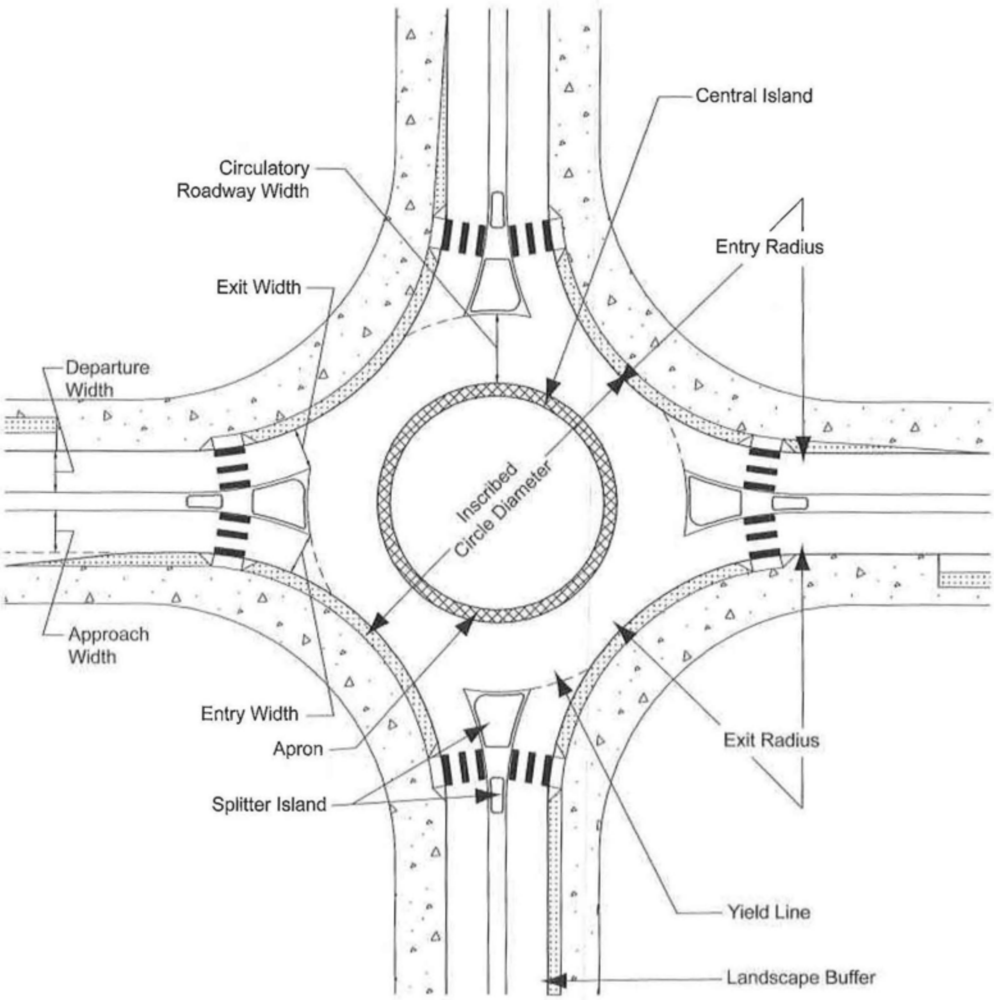


Figure 9-70. Basic Geometric Elements of a Roundabout

SOURCE: AASHTO Green Book

9.1 Characteristics

A. **Priority to Circulating Vehicles** – circulating vehicles have right-of-way.

B. **Yield-at-Entry.** Also known as "*off-side priority*" or "*yield-to-left*" rule. Yield signs used as entry control.

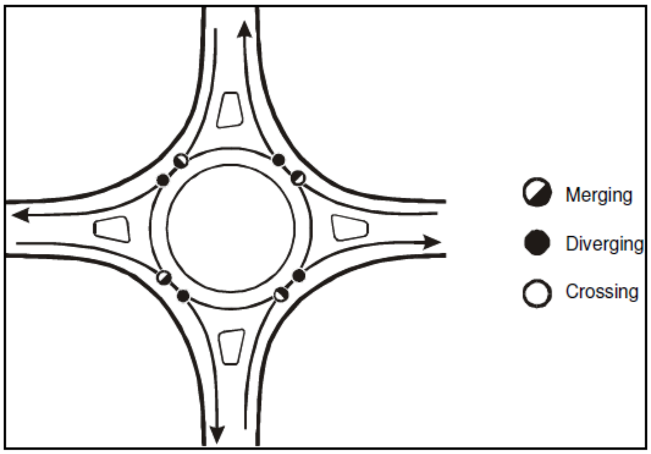
C. **Direction of Circulation**. Vehicles circulate counter-clockwise.

D. **Pedestrian Access** – allowed only across the legs of the roundabout behind the yield line

9.2 Sizes

- Mini-roundabouts – inscribed circle diameters as small as 15m [50 ft].
- Compact roundabouts – inscribed circle diameters between 30 and 35 m [98 to 115 ft]
- Large roundabouts – up to 150 m [492 ft] in diameter; often with multilane circulatory roadways and more than four entries.

9.3 Conflict Points



- A four-leg intersection would have 32 conflict points.
- A four-leg single-lane roundabout reduces the number of conflict points to only eight.
- A traditional signalized intersection has potential for four vehicle-pedestrian conflicts, all originating from different directions.
- Roundabouts require pedestrians to cross one direction of traffic at a time allowing the pedestrian to concentrate entirely on one direction of traffic with one conflict point at a time.

PROBLEM 40
A typical four-legged modern roundabout has a total of _____ vehicular conflict points.

A. 4.
B. 8
C. 16
D. 24.
E. 32.

SOLUTION 40
The correct answer is B.

10. Travel Demand Forecasting

- Attempts to quantify the amount of travel on a transportation system.
- Involves dividing urban areas into a series of zones
- Is created by the physical separation of urban activities.
- The supply of transportation is represented by the service characteristics of highway and transit networks.

Four basic phases of the travel-demand forecasting process:

1. **Trip generation** forecasts the number of trips that will be made.
2. **Trip distribution** determines where the trips will go.
3. **Mode Choice** predicts how the trips will be divided among the variable modes of travel.
4. **Trip assignment** (highway and transit) predicts the routes that the trips will take, resulting in traffic forecasts for the highway system and ridership forecasts for the transit system.

10.1 Trip Generation

A process by which measures of urban activity are converted into numbers of trips, e.g., number of trips generated by a shopping center is very different from the number of tips generated by an industrial complex that occupies the same amount of space.

PROBLEM 41

Estimate the number of trips per day generated by residential area containing 500 detached, single family homes. The ITE land use codes for single family homes is 210 (shown on the next page).

a. Using the **Regression Equation**, calculate the Average Daily Trip Generation

b. Using the **Average Trip Rate Method,** calculate the Average Daily Trip Generation.

SOLUTION 41

a. Using the Regression Equation,

$$Ln(T) = 0.920\ Ln(X) + 2.707 \qquad \text{Where} \qquad X = 500$$
$$Ln(T) = 0.920\ Ln(500) + 2.707$$
$$Ln(T) = 5.717 + 2.707 = 8.424;$$
$$T = e^{8.424} = 4{,}557 \text{ daily trips}$$

b. Use the Average Trip Rate Method,

T = trip rate x units
For land use code 210, the trip rate = 9.57 trips/unit.
T = 9.57 x 500 = 4,785 daily trips

Single-Family Detached Housing
(210)

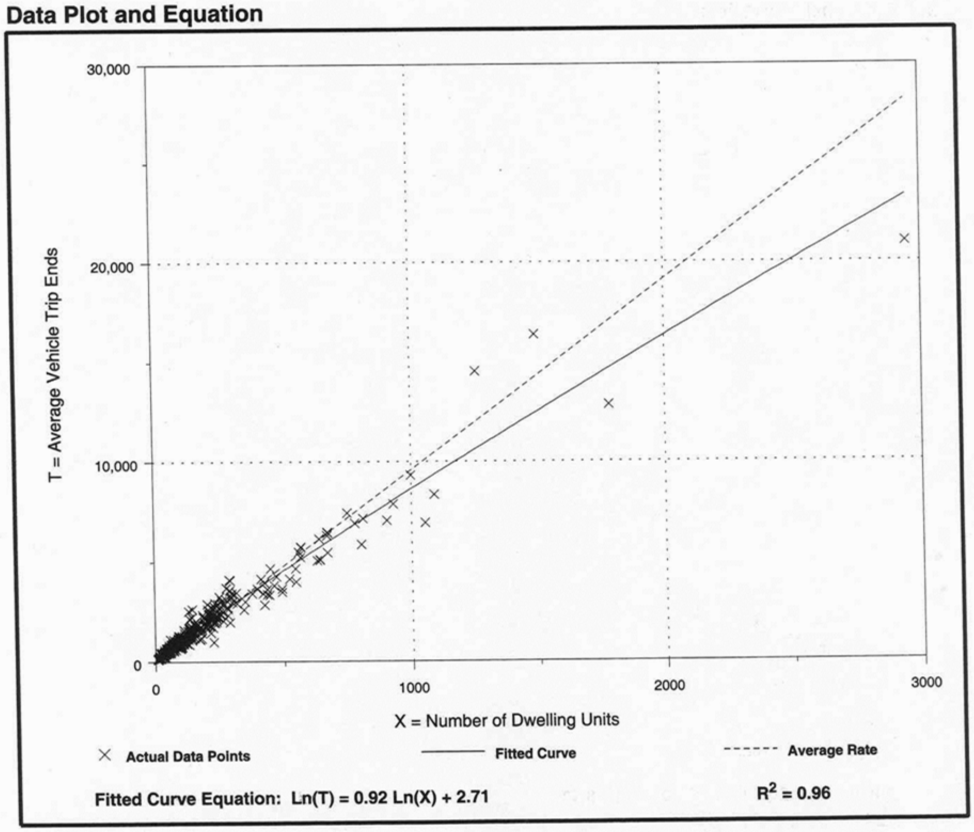
Average Vehicle Trip Ends vs: Dwelling Units
On a: Weekday

Number of Studies: 350
Avg. Number of Dwelling Units: 197
Directional Distribution: 50% entering, 50% exiting

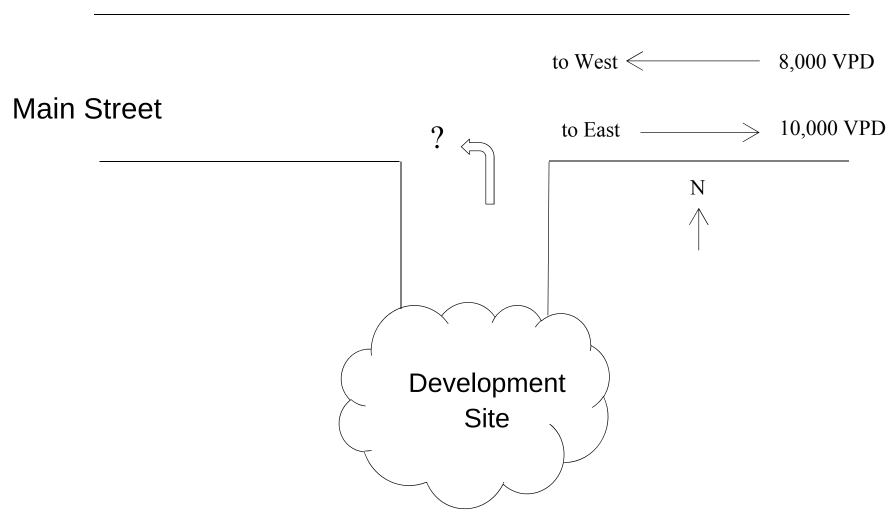
Trip Generation per Dwelling Unit

Average Rate	Range of Rates	Standard Deviation
9.57	4.31 - 21.85	3.69

Data Plot and Equation



PROBLEM 42
From Problem 1, based on the regression equation, it is estimated that 500 detached, single family homes would generate approximately 4,600 trips per day. Assume that this development is located on the south side of a major east-west road (Main Street) as shown below:



Find the number of vehicles in a day leaving the development turning to the left?

SOLUTION 42
From the ITE Exhibit provided in Problem 1, directional distribution of traffic is 50% entering and 50% exiting. Therefore, 50% of 4,600 trips would exit = 2,300 vpd would exit.

Out of 2,300 vpd, some would turn left and some would turn right. In other words, some would go west and some would go east. In order to split left and right traffic, we need to see the distribution of traffic on the Main Street.

On Main Street, traffic that goes east – 10,000 vpd
On Main Street, traffic that goes west – 8,000 vpd

In terms of percentage, traffic that goes east = 10,000/18,000 = 55.5 %
In terms of percentage, traffic that goes west = 8,000/18,000 = 44.5%

This means, out of 2,300 vpd exiting from the development, 55.5% would go east (right turn) and 44.5% would go west (left turn). Therefore, number of vehicles in a day leaving the development turning to the left = 44.5% * 2,300 vpd = 1,022 vpd

PROBLEM 43

A study determined that trips generation from a residential neighborhood is directly related to number of persons and number of autos per household. After conducting studies at different residential neighborhoods, the following relationship is established:

Number of trips generated per household per day = 0.44 + 1.6P + 2.1A

Where:

P = number of persons per household

A = number of autos per household

If a residential neighborhood contains 500 households with an average of 4.5 persons and 2.5 autos per household, how many trips this neighborhood is expected to generate per day?

SOLUTION 43

Number of trips generated per household per day = 0.44 + 1.6P + 2.1A

Number of trips generated by 500 households per day = 500 (0.44 + 1.6P + 2.1A)

= 500 (0.44+1.6*4.5 +2.1*2.5)

= 6,445

The correct Answer is B

10.2 Trip Distribution

After the trip-generation stage, the analyst knows the number of trip productions and trip attractions that each zone will have. The next step is to determine where do the attractions to a zone come from, and where do the trip productions go. More specifically, what are the zone-to-zone travel volumes? How many trips can be expected to be made between one zone and all the other zones? Trip distribution determines where the trips produced in each zone will go – how they will be divided among all other zones in the study area.

There are several methods of trip distribution analysis, including:

- Fratar Method (only method covered in this review)
- Intervening Opportunity Model
- Gravity Model

10.2.1 Fratar Method

This is based on the following assumptions:

1. The distribution of future trips from an origin is proportional to the present trip distribution
2. The future distribution is modified by the growth factor of the zone to which these trips are attached.

This model has been used extensively in several metropolitan areas, particularly for estimating external trips coming from outside the study areas to zones located within the study area.

$$t_{ij}^{f} = t_{ij}^{b} \frac{O_i^f}{O_i^b} \frac{D_j^f}{D_j^b} \frac{\sum_{k=1}^{n} t_{ik}^{b}}{\sum_{k=1}^{n} D_k^f}$$

Where :

O_i^f and O_i^b = future and base year origin trips from zone i

D_j^f and D_j^b = future and base year destination trips to zone j

t_{ij}^f and t_{ij}^b = future and base year trips from zone i to zone j

PROBLEM 44

An origin zone *i* with 20 base-year trips going to *j*, *k*, and *l* numbering 4, 6, and 10, respectively. Growth rates for *i*, *j*, *k*, and *l* are 2, 3, 4, and 6 respectively in 25 years. Determine the future trips from i to *j*, *k*, and *l* in the future year.

SOLUTION 44

$$t_{ij}^{b} = 4;\ t_{ik}^{b} = 6;\ t_{il}^{b} = 10$$

$$O_i^b = 20;\ O_i^f = 20 \times 2 = 40$$

$$D_j^b = 4;\ D_k^b = 6;\ D_l^b = 10$$

$D_j^f = 4 \times 3 \ = 12$

$D_k^f =$

$D_l^f =$

$\sum D^f = 96$

$$t_{ij}^{f} = \left(4 \times \frac{40}{20} \times \frac{12}{4}\right)\left(\frac{4+6+10}{96}\right) = 24 \times \frac{20}{96} \quad = 5$$

$$t_{ik}^{f} = \qquad = 48 \times \frac{20}{96} \quad = 10$$

$$t_{il}^{f} = \qquad = 120 \times \frac{20}{96} \quad = 25$$

$$Total \ = 40$$

11. Capacity Analysis

Reference: Highway Capacity Manual 2010

Capacity analysis is the study of various types of highway facilities and their ability to carry traffic.

Level-of-service Concept

Level-of-service (LOS) is a letter designation that describes a range of operating conditions on a particular type of facility.

LOS	Traffic Flow Condition
A	Free flow
B	Reasonably Free flow
C	Stable flow
D	Approaching unstable flow
E	Unstable flow
F	Forced or breakdown flow

See the LOS parameters (quantitative) list for different facilities in HCM Exhibit 2-2.

For design purpose, use the following guidelines based on AASHTO Green Book recommendations in Exhibit 2-5 based on area and terrain type:

Functional Class	Rural Level	Rural Rolling	Rural Mountainous	Urban and Suburban
Freeway	B	B	C	C or D
Arterial	B	B	C	C or D
Collector	C	C	D	D
Local	D	D	D	D

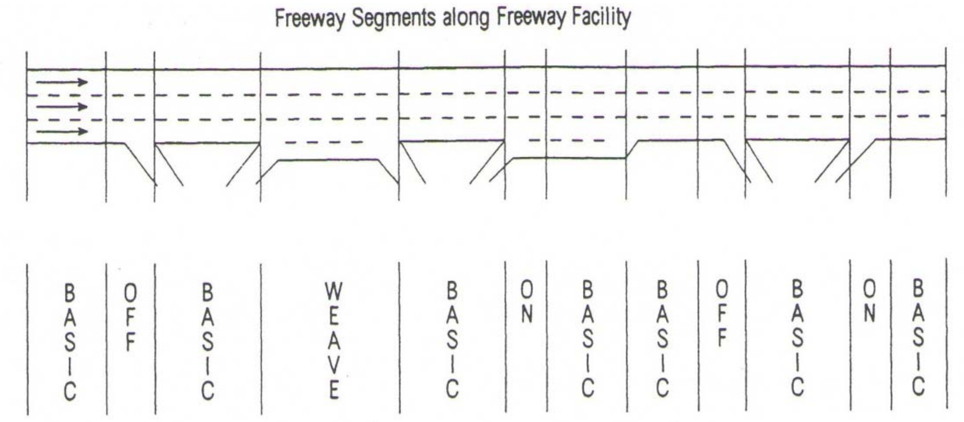
Capacity - *Capacity* (c) is the maximum "rate of flow" which the facility can accommodate.

Type of Facility	Capacity	Ideal Conditions
Freeways	2400 pcphpl	• Base Free flow speed (BFFS) $\geq$75.4 mph • 12 foot minimum lane widths • 6 foot minimum lateral clearance at right-side shoulder • 2 foot minimum lateral clearance at median side shoulder • Level terrain with grades no greater than 2 percent • Only passenger cars in the traffic stream • Driver population consisting of regular users of the facility

11.1 Freeways

Freeways include the following components:

- Basic Freeways
- Weaving Sections
- On-ramps
- Off-ramps



11.1.1 Basic Freeways

Reference - Chapter 11 of HCM

- Level of Service (LOS) criteria is **Density**
- Density = $\frac{v_p}{S}$

  Where: v_p = Passenger car equivalent flow rate (pcphpl)

  S = Free flow speed (FFS)

Step 1 – Compute Free Flow Speed (Equation 11-1)

Step 2 – Compute Flow Rate (Equations 11-2 and 11-3)

Step 3a – Determine Density using the above equation

Step 3b – Determine LOS (Exhibit 11-5 or the below table)

Table – LOS Criteria for Freeways

LOS	Density Range (pc/mi/ln)
A	≤ 11
B	>11-18
C	>18-26
D	>26-35
E	>35-45
F	>45

SOURCE: HCM Exhibit 11-5

PROBLEM 45

A 6-lane (3 lanes in each direction) freeway passes through level terrain in an urban area. The freeway is constructed with 11 ft lanes and concrete barriers 3 ft from the outer pavement edges of both outer lanes. The one-direction peak hourly volume during the weekday commute is 2200 vph. Traffic includes 4% buses, 6% trucks, and 2% recreational vehicles (RVs). There is one ramp per mile on average. The peak hour factor is 0.92. The posted speed limit is 55 mph.

(a) What is the passenger car equivalent flow rate per lane?
(b) What is the free flow speed?
(c) What is the density?
(d) What is the weekday peak-hour level of service?

SOLUTION 45

(a) $$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

Where: V=2200 vph; PHF=0.92; N = 3; f_{HV} = ?; f_p = ?

Trucks and buses have the same passenger car equivalents, so the "truck" fraction is 10% (4% + 6%). The traffic includes 2% RVs. From Exhibit 11-10 of HCM, for level terrain, E_T = , and E_R = ;Using Eq. 11-3

The commuter population knows the route, so f_P = 1.0

$$f_{HV} = \frac{1}{1+P_T(E_T-1)+P_R(E_R-1)} = \qquad = 0.9488$$

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p} = \frac{2200}{0.92 \times 3 \times 0.95 \times 1.0} = \frac{2200}{2.622} = 839 \text{ pcphpl}$$

(b)

FFS = 75.4 – f_{LW} – f_{LC} – 3.22 $TRD^{0.84}$ =
= 69.08 mph

(c) Density

$$D = \frac{v_p}{S} = \qquad = 12.15\ pcpmpl$$

(d) Using LOS Criteria table, 11 < 12.15 < 18;
Therefore the roadway is operating at LOS B

11.1.1.1 Volume Parameters

NOTE: Part of this section is reviewed under Geometrics section of the regular classes also. This is repeated here to have a quick overview before going into the depth problems.

- AADT – Average Annual Daily Traffic (one-way or two-way)
  AADT = Total Yearly Volume/365
  AADT = 24-hour count on a particular day * daily variation factor * monthly variation factor
- DHV – Design Hour Volume – 30th highest hourly volume expected over an year (two-way unless noted)
- K or K- factor = $\frac{\text{DHV}}{\text{AADT}}$ (or) DHV = K-factor * AADT
- D or D-factor – Directional factor = $\frac{\text{Peak Direction Hourly Volume}}{\text{Two - way Hourly Volume}}$
- DDHV – Directional Design Hour Volume = D * DHV = D * K * AADT (HCM Eqn. 11-8)
- Demand Flow Rate (pc/h/ln) = $v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$ (HCM Eqn. 11-2)
- Service Flow Rate (veh/h) = $SF = MSF \times N \times f_{HV} \times f_p$ (HCM Eqn. 11-9)
- Maximum Service Flow Rate = MSF (See Exhibit 11-17 . Values under LOS E are the capacities)
- Service Volume (veh/h) = SV = $SF \times PHF$ (HCM Eqn. 11-10)
- Daily Service Volume (veh/day) = DSV = $\frac{SV}{K \times D}$ (HCM Eqn. 11-11)

NOTE: SF and SV are stated for a single direction of the freeway per hour. DSV are stated as total volumes in both directions per day.

PROBLEM 46

For a basic freeway under the following conditions, estimate service flow rate and service volume:

- Number of lanes in each direction – 3
- Free Flow Speed = 70 mph
- Level of Service – E
- Heavy Vehicles Factor = 0.9
- Peak Hour Factor = 0.9
- Commute Traffic

SOLUTION 46
Service Flow Rate (veh/h) = $SF = MSF \times N \times f_{HV} \times f_p$

MSF = 2,400 pc/h/ln per Exhibit 11-17 for LOS E

For commute traffic, $f_p = 1.0$

$SF = 2{,}400 \times 3 \times 0.9 \times 1.0 = 6{,}480\ veh/h$

$SV = SF \times PHF = \ 6{,}480 \times 0.9 = 5{,}832\ veh/h$

PROBLEM 47
In how many years a six lane basic freeway segment will reach its capacity?

The Facts
- Existing Volume of 5,600 veh/h in one direction.
- Three lanes in each direction
- FFS = 70 mph
- Heavy Vehicles Factor = 0.952
- Peak Hour Factor = 0.95
- Commuter Traffic (regular users); and
- Traffic growth rate = 4% per year

SOLUTION 47
Step 1 – Find out SF at its capacity
$SF = MSF \times N \times f_{HV} \times f_p$

MSF at LOS E is synonymous with capacity
MSF = 2,400 pc/h/ln (per HCM Exhibit 11-17)

$SF = 2{,}400 \times 3 \times 0.952 \times 1.0 = 6{,}854\ veh/h$

Step 2 – Find out service volume
Service Volume (SV) at LOS E.

$SV = SF \times PHF = \ 6{,}854 \times 0.95 = 6{,}511\ veh/h$

6,511 veh/h is the maximum number of vehicles this basic freeway segment can carry under the given conditions. Therefore, 6,511 veh/ is the capacity of this segment.

Step 3 – Find out when it will reach the capacity

Given:

- Current traffic = 5,600 veh/h
- Capacity for the given conditions = 6,511 veh/h
- Growth rate = 4% per year

$5{,}600\ (1.04)^n = 6{,}511$

n =3.85 years

In approximately 4 years, this freeway will reach its capacity for the given conditions.

PROBLEM 48

A roadway is currently carrying an average daily traffic of 10,000. It is expected the traffic on this roadway will grow 8% per year for the next 15 years. How much traffic this roadway is expected to carry in 15 years?

SOLUTION 48

$$\begin{aligned}\text{ADT in 15 years} &= \text{Current ADT}\ (1+0.08)^{15}\\ &= 10{,}000\ (1.08)^{15}\\ &= 31{,}722\end{aligned}$$

11.2 Multi-lane Highways

Reference - Chapter 14 of HCM

How to differentiate Freeway vs Multi-lane Highway problem?

- CLUE 1 – If the problem states Freeway, then it is a Freeway problem (Chapter 11)
- CLUE 2 – If the problem states multi-lane, but the facility can be accessed only through interchanges, then it is a Freeway problem (Chapter 11)
- CLUE 3 – If the problem states multi-lane, but the facility can be accessed through intersections or driveways, then it is a Multi-lane Highway problem (Chapter 14)
- CLUE 4 – Freeways are always divided. Multi-lane Highways can be divided or undivided. If the problem states “undivided” highway, then it is a Multi-lane Highway problem (Chapter 14)

- Level of Service (LOS) criteria is **Density**
- Density = $\frac{v_p}{S}$

Where: v_p = Passenger car equivalent flow rate (pcphpl)

S = Free flow speed (FFS)

Table – LOS Criteria for Multi-lane Highways

LOS	FFS (mi/h)	Density (pc/mi/ln)
A	All	>0–11
B	All	>11–18
C	All	>18–26
D	All	>26–35
E	60	>35–40
	55	>35–41
	50	>35–43
	45	>35–45
F	Demand Exceeds Capacity	
	60	>40
	55	>41
	50	>43
	45	>45

SOURCE: HCM Exhibit14-4

Step 1 – Compute Free Flow Speed (Equation 14-1 & 14-2)
Step 2 – Compute Flow Rate (Equations 14-3 and 14-4)
Step 3a – Determine Density using the above equation
Step 3b – Determine LOS (Exhibit 14-4 or the above table)

PROBLEM 49

A 2.5-mi undivided 4-lane highway is located on level terrain. Determine the peak hour LOS for the highway? The Facts:

- 46.0 mi/h field-measured FFS
- 11-ft lane width
- 1,900 veh/h peak-hour-volume in one direction
- 13 percent trucks and buses,
- 2 percent RVs, and
- 0.90 PHF; and commuter traffic

SOLUTION 49

Step 1 – Compute Free Flow Speed – Given as 46 mi/h

Step 2 – Convert volume to flow rate

$$v_p = \frac{V}{(PHF)(N)(f_{HV})(f_p)}$$

Where: V = 1,900 vph; PHF = 0.90; N = 2; $f_p = 1.0$

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)} = \frac{1}{1 + 0.13(1.5 - 1) + 0.02(1.2 - 1)}$$

=0.935

$v_p = 1{,}129$ pcphpl

Step 3 – Determine LOS-> Density = $\frac{1129}{46}$ = 24.5; LOS = C

11.3 Pedestrian Facilities

Reference - Chapter 23 of HCM

- Level of Service (LOS) criteria for Walkways and Sidewalks is primarily **Flow Rate (p/min/ft)**

Step 1 – Compute Effective Walkway Width (Equation 23-1) by using Exhibits 23-10 and 23-11

Step 2 – Compute Pedestrian Flow Rate in Peak Hour (Equation 23-3)

Step 3a – Determine LOS for Average Flow (Exhibit 23-1 or the above table)

Step 3b – Determine LOS for Platoon Adjusted (Exhibit 23-2 or the above table)

Table – LOS Criteria for Walkways and Sidewalks

LOS	Flow Rate for Average Flow Criteria (p/min/ft)	Flow Rate for Platoon-Adjusted Flow Criteria (p/min/ft)
A	≤ 5	≤ 0.5
B	>5-7	>0.5-3
C	>7-10	>3-6
D	>10-15	>6-11
E	>15-23	>11-18
F	variable	>18

SOURCE: HCM Exhibits 23-1 and 23-2

PROBLEM 50

A 14.0-ft wide sidewalk is bordered by curb on one side and building face with window display on the other. What is the LOS during the peak 15 min on the average and within platoons.

Facts:

- 15-min peak flow rate – 1,250 p/h;
- No other obstructions

SOLUTION 50

11.4 Signalized Intersections

Cycle length – the time required for one complete sequence of all signal indications

Phase – the right-of-way (green), change (yellow), and clearance (all red) intervals in a cycle that are assigned to an independent traffic movement or combination of movements

Green interval – the right-of-way interval during which the signal indication is green

Yellow Change interval – the first interval following the green interval or which the signal indication is yellow

Clearance interval – an interval that follows a yellow change interval and precedes the next conflicting green interval

11.4.1 Change Interval

Reference – Any Traffic Engineering Book

- Also known as "Yellow Interval"

$$y = t + \frac{v}{2a + 2Gg}$$

Where: y = length of yellow interval (sec)
t = driver perception/reaction time (1.0 sec generally used)
v = velocity of approaching vehicle (fps)
a = deceleration rate (10 fps^2 generally used)

G = acceleration due to gravity (32.2 fps^2 generally used)
g = grade of approach (percent/100)

11.4.2 Clearance Interval

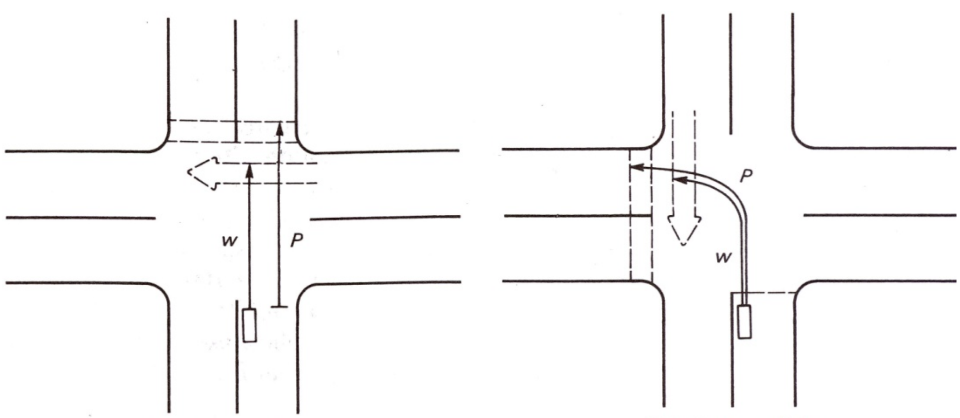
Reference – Any Traffic Engineering Book

- Also known as "Red Clearance Interval" and "All Red"

$$r = \frac{w + L}{v}$$ - If there is no pedestrian traffic

$$r = \frac{P + L}{v}$$ - If there is pedestrian traffic or the crosswalk is protected by ped. signal

Where: r = length of red clearance interval (sec)
w = width of intersection; more precisely, the length of the vehicle path from the departure stop line to the far side of farthest conflicting traffic lane (ft)
P = width of intersection; more precisely, the length of the vehicle path from the departure stop line to the far side of farthest conflicting ped crosswalk (ft)
L = Length of vehicle (20 ft generally used)
v = speed of vehicle through intersection (fps)



(a) Through vehicle (b) Left-turn vehicle

PROBLEM 51
Estimate change intervals for grades -2%, 0%, and +2% for an approach speed of 30 mph.

SOLUTION 51

- For -2% grade,

$$y = \qquad = 3.4 \text{ sec}$$

- For 0% grade, y = 3.2 sec
- For +2% grade, y = 3.1 sec

11.4.3 Cycle Length, Phases, Green Interval

Reference - Chapter 18 of HCM

Using Equation 18-17 (modified) from HCM, required Cycle Length for an intersection can be computed as follows:

$$C = \frac{L * X_c}{X_c - \sum y_i}$$

Where,

$$\sum y_i = (v/s)_1 + (v/s)_2 + (v/s)_3 + ...(v/s)_n$$

PROBLEM 52
The traffic volumes for a two phase signal at the intersection of High Street and Broad Street are summarized below.

Street	Flow Rate	Max Saturation Flow
High St.	500 vph	1500 vph
Broad St.	600 vph	1200 vph

Given the following facts, answer parts 'a' through 'e'

- lost time/ phase: 4 sec; desired intersection v/c (X_c) = 0.90

SOLUTION 52

a.) What is the actual volume-to-saturation flow ratio for High St.?

v/s = 500/1500 = **0.33**

b.) What is the actual volume-to-saturation flow ratio for Broad St.?

v/s = 600/1200 = **0.50**

c.) What cycle length, C, should be used?

Total Lost time: L =

$$C = \frac{L \times X_c}{[X_c - (v/s)_1 - (v/s)_2]} = \qquad = 103\ sec$$

d.) What is the optimum (effective) green time that will maintain the optimum v/c ratio on each street?

$$g_{High\ St} = \left(\frac{v}{s}\right)\left(\frac{C}{X_i}\right) = 0.33\left(\frac{103}{0.9}\right) = 38\ sec$$

$$g_{Broad\ St} = \left(\frac{v}{s}\right)\left(\frac{C}{X_i}\right) = 0.50\left(\frac{103}{0.9}\right) = 57\ sec$$

e.) What is the phase length for each street?

Phase length = Effective Green Time of the Phase + Lost Time of the Phase

Phase length for High Street = 38 sec + 4 sec = 42 sec
Phase length for Broad Street = 57 sec + 4 sec = 61 sec

Clue: When we add all phase lengths, it should equal to Cycle length. In this example, 42 sec plus 61 sec = 103 sec which is the Cycle Length.

f.) What is the minimum cycle length, C, should be used? (Use X_c = 1.00)

$$C = \frac{L \times X_c}{[X_c - (v/s)_1 - (v/s)_2]} = \qquad = 47\ sec$$

PROBLEM 53
At a 90 second cycle four-legged signalized intersection, one lane southbound approach has a 30 second green time, a 3 second yellow time, and 1 second all red time. The intersection has saturation headway of 2 seconds per vehicle, start-up lost time of 2 seconds per phase, and clearance lost time of 3 seconds per phase. Under these conditions, what would be the approximate capacity of this southbound approach?

SOLUTION 53
Using HCM Equation 18-15 on page 18-41,

$$c = Ns\frac{g}{C}$$

Where
c = Capacity in vph
N = number of lanes = 1
S = saturation flow rate = $\frac{3{,}600\ sec/hr}{Satudation\ Headway} = \frac{3{,}600\ sec/hr}{2\ sec/veh} = 1{,}800\ vph$
g = effective green time = Phase Length – Lost Time = (30+3+1)–(2+3) = 29 sec
C = Cycle length = 90 sec

$$c = Ns\frac{g}{C} = 1 * 1{,}800 * \frac{29}{90} = \ 580\ vph$$

11.4.4 Level of Service Analysis
Reference - Chapter 18 of HCM

- Level of Service (LOS) criteria is **Control Delay per Vehicle (sec)**

Table – LOS Criteria for Signalized Intersections

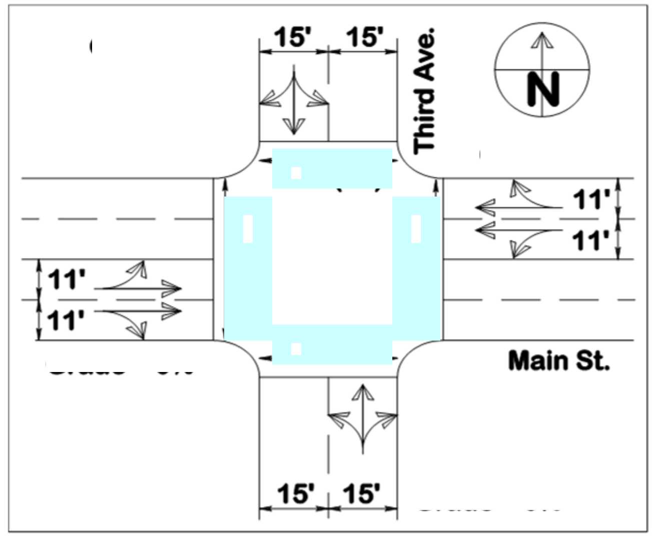
Control Delay (s/veh)	LOS by Volume-to-Capacity Ratio[a] ≤1.0	>1.0
≤10	A	F
>10–20	B	F
>20–35	C	F
>35–55	D	F
>55–80	E	F
>80	F	F

Note: [a] For approach-based and intersectionwide assessments, LOS is defined solely by control delay.

SOURCE: HCM Exhibit 18-4

PROBLEM 54

The intersection of Third Avenue (NB/SB) and Main Street (EB/WB) is located in the Central Business District (CBD) of a small urban area. Intersection geometry is shown. What is the minimum green time required for pedestrian crossings?



The Facts:

- Cycle Length = 60 sec
- Two phase signal
- Crosswalk width = 10'; Pedestrian Volume = 120 p/h at each approach

SOLUTION 54

Minimum effective green time required for pedestrians can be calculated using the following equation (Equation 18-66 of HCM): $3.2 + \frac{L}{S_p} + 0.27N_{ped}$

Using Eqn 18-54,

Pedestrians per cycle (N_{ped}) = = 2 ped/cycle

- to cross Third Ave. = $3.2 + \frac{L}{S_p} + 0.27N_{ped}$

  $=$ $= 11.24\ sec$

- to cross Main Street = $3.2 + \frac{L}{S_p} + 0.27N_{ped}$

  $= 3.2 + \frac{44}{4.0} + 0.27 \times 2 = 14.74\ sec$

PROBLEM 55

Given the following information by approach, what is the intersection LOS?

- EB – Flow rate 800 veh/h; Approach Control Delay 59.4 sec/veh
- WB – Flow rate 833 veh/h; Approach Control Delay 31 sec/veh
- NB – Flow rate 466 veh/h; Approach Control Delay 14.4 sec/veh
- SB – Flow rate 667 veh/h; Approach Control Delay 21.9 sec/veh

SOLUTION 55

Intersection Delay as per equation 18-48 of HCM ,

$$d_I = \frac{\sum d_A v_A}{\sum v_A} =$$

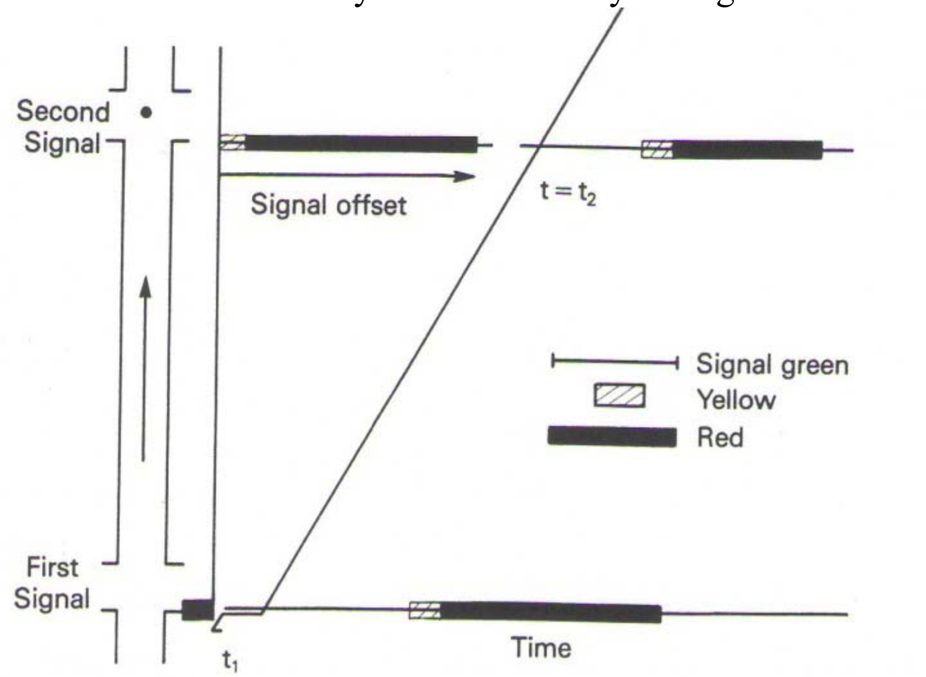
$$= 34.2\ sec/veh$$

From Exhibit 18-4 of HCM, LOS = C

12. Signal Coordination

Reference: Any Traffic Engineering book

In situations where signals are relatively closely spaced, it is necessary to coordinate their green times so that vehicles may move efficiently through the set of signals.



Time-Space Diagram – It is simply the plot of signal indications as a function of time for two or more signals. In the above figure, time-space diagram is shown for two signals. In this figure, time is shown on the X-axis and signal spacing is shown in the Y-axis. It is also acceptable to show signal spacing on the X-axis and time on the Y-axis.

Upstream and Downstream Signals - In the above figure, traffic is moving from the First Signal to the Second Signal. In this case, First Signal is called Upstream signal and Second Signal is called Downstream signal.

Signal Offset – The difference between the green initiation times between successive signals.

Platoon – A platoon is a group of vehicles moving along a facility together, with significant gaps between one such group and the next. On signalized facilities, these platoons are formed by the pattern of green phases at successive intersections.

PROBLEM 56

Consider a one-way arterial as shown in the following figure with the indicated spacing between four signals. Assuming there are no vehicles queued at the signals, determine the offsets between the signals if the desired platoon speed is 50 fps and cycle length 60 seconds.

SOLUTION 56

Signal (Downstream)	Relative to Signal (Upstream)	Ideal Offset
2	1	1000/50 = 20 sec
3	2	
4	3	

PROBLEM 57

Consider a one-way arterial as shown in the following figure with the indicated spacing between five signals. Assuming there are no vehicles queued at the signals, at what speed a vehicle should be travelling to avoid stopping at signal 2 after leaving signal 3. Given the following parameters:

Offset between Signals 3 and 2 = 60 sec
Cycle length = 90 sec

SOLUTION 57

13. Traffic Control Devices

Reference: Manual of Uniform Traffic Control Devices, 2009 Edition

13.1 Signs

13.1.1 Sign Colors

Reference: Table 2A-5 in Manual of Uniform Traffic Control Devices, 2009 Edition

2009 Edition Page 33

Table 2A-5. Common Uses of Sign Colors

Type of Sign	Legend								Background										
	Black	Green	Red	White	Yellow	Orange	Fluorescent Yellow-Green	Fluorescent Pink	Black	Blue	Brown	Green	Orange*	Red*	White	Yellow*	Purple	Fluorescent Yellow-Green	Fluorescent Pink
Regulatory	X		X	X					X					X	X				
Prohibitive			X	X										X	X				
Permissive		X													X				
Warning	X															X			
Pedestrian	X															X		X	
Bicycle	X															X		X	
Guide				X								X							
Interstate Route				X						X				X					
State Route	X														X				
U.S. Route	X														X				
County Route					X					X									
Forest Route				X							X								
Street Name				X								X							
Destination				X								X							
Reference Location				X								X							
Information				X						X		X							
Evacuation Route				X						X									
Road User Service				X						X									
Recreational				X							X	X							
Temporary Traffic Control	X												X						
Incident Management	X												X						X
School	X																	X	
ETC-Account Only	X																X****		
Changeable Message Signs																			
Regulatory			X***	X					X										
Warning					X				X										
Temporary Traffic Control					X	X			X										
Guide				X					X			X**							
Motorist Services				X					X	X**									
Incident Management					X			X	X										
School, Pedestrian, Bicycle					X		X		X										

* Fluorescent versions of these background colors may also be used.

** These alternative background colors would be provided by blue or green lighted pixels such that the entire CMS would be lighted, not just the legend.

*** Red is used only for the circle and slash or other red elements of a similar static regulatory sign.

**** The use of the color purple on signs is restricted per the provisions of Paragraph 1 of Section 2F.03.

13.1.2 Warning Signs

Table 2C-4. Guidelines for Advance Placement of Warning Signs

Posted or 85th-Percentile Speed	Advance Placement Distance[1]								
	Condition A: Speed reduction and lane changing in heavy traffic[2]	Condition B: Deceleration to the listed advisory speed (mph) for the condition							
		0[3]	10[4]	20[4]	30[4]	40[4]	50[4]	60[4]	70[4]
20 mph	225 ft	100 ft[6]	N/A[5]	—	—	—	—	—	—
25 mph	325 ft	100 ft[6]	N/A[5]	N/A[5]	—	—	—	—	—
30 mph	460 ft	100 ft[6]	N/A[5]	N/A[5]	—	—	—	—	—
35 mph	565 ft	100 ft[6]	N/A[5]	N/A[5]	N/A[5]	—	—	—	—
40 mph	670 ft	125 ft	100 ft[6]	100 ft[6]	N/A[5]	—	—	—	—
45 mph	775 ft	175 ft	125 ft	100 ft[6]	100 ft[6]	N/A[5]	—	—	—
50 mph	885 ft	250 ft	200 ft	175 ft	125 ft	100 ft[6]	—	—	—
55 mph	990 ft	325 ft	275 ft	225 ft	200 ft	125 ft	N/A[5]	—	—
60 mph	1,100 ft	400 ft	350 ft	325 ft	275 ft	200 ft	100 ft[6]	—	—
65 mph	1,200 ft	475 ft	450 ft	400 ft	350 ft	275 ft	200 ft	100 ft[6]	—
70 mph	1,250 ft	550 ft	525 ft	500 ft	450 ft	375 ft	275 ft	150 ft	—
75 mph	1,350 ft	650 ft	625 ft	600 ft	550 ft	475 ft	375 ft	250 ft	100 ft[6]

[1] The distances are adjusted for a sign legibility distance of 180 feet for Condition A. The distances for Condition B have been adjusted for a sign legibility distance of 250 feet, which is appropriate for an alignment warning symbol sign. For Conditions A and B, warning signs with less than 6-inch legend or more than four words, a minimum of 100 feet should be added to the advance placement distance to provide adequate legibility of the warning sign.

[2] Typical conditions are locations where the road user must use extra time to adjust speed and change lanes in heavy traffic because of a complex driving situation. Typical signs are Merge and Right Lane Ends. The distances are determined by providing the driver a PRT of 14.0 to 14.5 seconds for vehicle maneuvers (2005 AASHTO Policy, Exhibit 3-3, Decision Sight Distance, Avoidance Maneuver E) minus the legibility distance of 180 feet for the appropriate sign.

[3] Typical condition is the warning of a potential stop situation. Typical signs are Stop Ahead, Yield Ahead, Signal Ahead, and Intersection Warning signs. The distances are based on the 2005 AASHTO Policy, Exhibit 3-1, Stopping Sight Distance, providing a PRT of 2.5 seconds, a deceleration rate of 11.2 feet/second^2, minus the sign legibility distance of 180 feet.

[4] Typical conditions are locations where the road user must decrease speed to maneuver through the warned condition. Typical signs are Turn, Curve, Reverse Turn, or Reverse Curve. The distance is determined by providing a 2.5 second PRT, a vehicle deceleration rate of 10 feet/second^2, minus the sign legibility distance of 250 feet.

[5] No suggested distances are provided for these speeds, as the placement location is dependent on site conditions and other signing. An alignment warning sign may be placed anywhere from the point of curvature up to 100 feet in advance of the curve. However, the alignment warning sign should be installed in advance of the curve and at least 100 feet from any other signs.

[6] The minimum advance placement distance is listed as 100 feet to provide adequate spacing between signs.

PROBLEM 58
A two-lane rural highway has a posted speed of 55 mph. In one area of the roadway there is a horizontal curve with an advisory speed of 30 mph. Based on this advisory speed, where should an advanced placement warning sign be located with respect to the point of curvature?

SOLUTION 58
200 feet - See Table 2C-4 in MUTCD

13.1.3 Guide Sign Minimum Letter Size

- To determine sign sizes
- To make sure it is visible enough based on type of roads and speeds
- See Section 2D.06 of MUTCD for Conventional Roads
- See Tables 2E-2 and 2E-3 of MUTCD for Expressways
- See Tables 2E-4 and 2E-5 of MUTCD for Freeways

PROBLEM 59
You have assigned responsibility for designing new guide sign for a major interchange of a freeway with a high-volume, multi-lane highway. What is the minimum size (in inches) of upper-case letters to be used for the names of destination shown on sign legends.

SOLUTION 59
For Freeway, see Table 2E-4. Per Section 2E-32 (page 216), this is Major interchange Category ‘b’. For name of destination sign, the minimum size of upper-case letters is 20 inches.

13.2 Delineators

Reference: See Chapter 3F in Manual of Uniform Traffic Control Devices, 2009 Edition (See Pages 425 and 427)

Table 3F-1. Approximate Spacing for Delineators on Horizontal Curves

Radius (R) of Curve	Approximate Spacing (S) on Curve
50 feet	20 feet
115 feet	25 feet
180 feet	35 feet
250 feet	40 feet
300 feet	50 feet
400 feet	55 feet
500 feet	65 feet
600 feet	70 feet
700 feet	75 feet
800 feet	80 feet
900 feet	85 feet
1,000 feet	90 feet

Notes:
1. Spacing for specific radii may be interpolated from table.
2. The minimum spacing should be 20 feet.
3. The spacing on curves should not exceed 300 feet.
4. In advance of or beyond a curve, and proceeding away from the end of the curve, the spacing of the first delineator is 2S, the second 3S, and the third 6S, but not to exceed 300 feet.
5. S refers to the delineator spacing for specific radii computed from the formula $S=3\sqrt{R-50}$.
6. The distances for S shown in the table above were rounded to the nearest 5 feet.

13.3 Intersection Signal Warrant Analysis

MUTCD Chapter 4C (page 436) identifies nine types of signal warrants.

Warrant 1, Eight-Hour Vehicular Volume
Warrant 2, Four-Hour Vehicular Volume
Warrant 3, Peak Hour
Warrant 4, Pedestrian Volume
Warrant 5, School Crossing
Warrant 6, Coordinated Signal System
Warrant 7, Crash Experience
Warrant 8, Roadway Network
Warrant 9, Intersection Near a Grade Crossing

PROBLEM 60

Two streets having the traffic volumes indicated below, intersect at a 4-way intersection. Each street has one approaching lane to the intersection. Determine for how many hours intersection has met the Condition A criteria based on the eight-hour vehicular volume warrant analysis.

Major Street Volumes

Hour Begins	EB	WB	Hour Begins	EB	WB
6: a.m	390	100	12:pm	500	500
7: a.m	400	90	1:pm	345	400
8: a.m	600	400	2:pm	300	350
9: a.m	500	350	3:pm	350	600
10: a.m	400	400	4:pm	400	700
11: a.m	450	500	5:pm	300	600

Minor Street Volumes

Hour Begins	NB	SB	Hour Begins	NB	SB
6: a.m	35	60	12:pm	175	150
7: a.m	100	170	1:pm	125	160
8: a.m	100	190	2:pm	80	140
9: a.m	80	90	3:pm	135	105
10: a.m	90	110	4:pm	200	175
11: a.m	80	120	5:pm	180	195

SOLUTION 60

Minimum Vehicle Warrant Criteria		
Major Street		Minor Street
• At least 500 vph • Both directions Combined • 8 of the hours in the day	And	• At least 150 vph • Either direction • 8 of the hours in day
o Same 8 hours for major and minor street		

Major Street Volumes									
Hour Begins	**EB**	**WB**	**Total**		**Hour Begins**	**EB**	**WB**	**Total**	
6: a.m	390	100	490		12:pm	500	500	1000	
7: a.m	400	90	490		1:pm	345	400	745	
8: a.m	600	400	1000		2:pm	300	350	650	
9: a.m	500	350	850		3:pm	350	600	950	
10: a.m	400	400	800		4:pm	400	700	1100	
11: a.m	450	500	950		5:pm	300	600	900	
The major street meets the criteria during , therefore the major street meets the signal warrant criteria.									

Minor Street Volumes							
Hour Begins	**NB**	**SB**		**Hour Begins**	**NB**	**SB**	
6: a.m	35	60		12:pm	175	150	◄
7: a.m	100	170	◄	1:pm	125	160	◄
8: a.m	100	190	◄	2:pm	80	140	
9: a.m	80	90		3:pm	135	105	
10: a.m	90	110		4:pm	200	175	◄
11: a.m	80	120		5:pm	180	195	◄
The minor road has 150 vph in only 6 hours, 6 <8, Therefore it does not meet the minor road criteria, and a signal is **NOT warranted** at this intersection (based on traffic data)							

Hours met for both Major and Minor Streets – 5 hours (8 am; 12 pm, 1 pm, 4 pm, and 5 pm)

13.4 Temporary Traffic Control Zone

Reference: MUTCD Chapter 6 (2009 Edition)

13.4.1 Transition Tapers

See Figure 6C-2 to apply the dimensions in the following tables

Table 6C-3. Taper Length Criteria for Temporary Traffic Control Zones

Type of Taper	Taper Length
Merging Taper	at least L
Shifting Taper	at least 0.5 L
Shoulder Taper	at least 0.33 L
One-Lane, Two-Way Traffic Taper	50 feet minimum, 100 feet maximum
Downstream Taper	50 feet minimum, 100 feet maximum

Note: Use Table 6C-4 to calculate L

Table 6C-4. Formulas for Determining Taper Length

Speed (S)	Taper Length (L) in feet
40 mph or less	$L = \frac{WS^2}{60}$
45 mph or more	$L = WS$

Where: L = taper length in feet
W = width of offset in feet
S = posted speed limit, or off-peak 85th-percentile speed prior to work starting, or the anticipated operating speed in mph

PROBLEM 61

There is shoulder work that needs to be completed on a rural two-lane highway. The speed limit is 50 MPH and the offset requires is 6 feet. What is the most appropriate taper length to place cones for this work zone?

SOLUTION 61

L = WS = 6 x 50 = 300 ft

Taper length = 0.33 L = 0.33 x 300 = 100 ft

13.4.2 Sign Locations

See Tables 6H-2 through 6H-4 and Figures 6H-1 through 6H-46 to determine location of temporary traffic control signs.

PROBLEM 62

There is shoulder work that needs to be completed on a rural two-lane highway. The 85^{th} percentile speed is 50 MPH and the offset required is 6 ft. What are the distances required to place "Shoulder Work" and "Road Work X Miles" signs in advance of the shoulder work?

SOLUTION 62

See Tables 6H-3, 6H-4, and Figure 6H-3.

L = WS = 6 * 50 = 300 ft

Distance to place "Shoulder Work" sign = 1/3 L + A = 1/3*300+500
= 600 ft

Distance to place "Road Work X Miles" sign = 1/3 L + A + B = 1/3*300+500+500
= 1,100 ft

PROBLEM 63

There is a road closure and diversion on a two-lane high speed urban highway. The 85^{th} percentile speed is 45 MPH. How far in advance of the diversion point should a driver see the very first sign about the road work?

SOLUTION 63

See Table 6H-3 and Figure 6H-7.

Distance to first sign from the diversion = A+B+C = 350+350+350 = 1,050 ft

PROBLEM 64

There is a left lane closure at the intersection of a four lane rural highway with a two lane highway. The 85th percentile speed is 45 MPH on the four-lane highway with 12 feet of offset. How far in advance of the first cone, where transition starts within the turn lane, should a driver see the first "Road Work Ahead" sign?

SOLUTION 64

See Table 6H-3 and Figure 6H-23.

Distance to "Road Work Ahead" sign from the first cone = A+B+C = 500+500+500 = 1,500 ft

14. Roadside Design

Reference: Roadside Design Guide, 4th Edition, 2011 by AASHTO

14.1 Roadside Clear Zone

See the table below (Refer to Table 3.1 in Chapter 3 of the Guide):

If the roadway is on an embankment (or on fill slope), refer to the columns under the heading "FORESLOPES". If the roadway is in cutting slope, refer to the columns under the heading "BACKSLOPES".

U.S. Customary Units

Design Speed (mph)	Design ADT	Foreslopes			Backslopes		
		1V:6H or flatter	1V:5H to 1V:4H	1V:3H	1V:3H	1V:5H to 1V:4H	1V:6H or flatter
≤40	UNDER 750[c]	7–10	7–10	[b]	7–10	7–10	7–10
	750–1500	10–12	12–14	[b]	12–14	12–14	12–14
	1500–6000	12–14	14–16	[b]	14–16	14–16	14–16
	OVER 6000	14–16	16–18	[b]	16–18	16–18	16–18
45–50	UNDER 750[c]	10–12	12–14	[b]	8–10	8–10	10–12
	750–1500	14–16	16–20	[b]	10–12	12–14	14–16
	1500–6000	16–18	20–26	[b]	12–14	14–16	16–18
	OVER 6000	20–22	24–28	[b]	14–16	18–20	20–22
55	UNDER 750[c]	12–14	14–18	[b]	8–10	10–12	10–12
	750–1500	16–18	20–24	[b]	10–12	14–16	16–18
	1500–6000	20–22	24–30	[b]	14–16	16–18	20–22
	OVER 6000	22–24	26–32[a]	[b]	16–18	20–22	22–24
60	UNDER 750[c]	16–18	20–24	[b]	10–12	12–14	14–16
	750–1500	20–24	26–32[a]	[b]	12–14	16–18	20–22
	1500–6000	26–30	32–40[a]	[b]	14–18	18–22	24–26
	OVER 6000	30–32[a]	36–44[a]	[b]	20–22	24–26	26–28
65–70[d]	UNDER 750[c]	18–20	20–26	[b]	10–12	14–16	14–16
	750–1500	24–26	28–36[a]	[b]	12–16	18–20	20–22
	1500–6000	28–32[a]	34–42[a]	[b]	16–20	22–24	26–28
	OVER 6000	30–34[a]	38–46[a]	[b]	22–24	26–30	28–30

PROBLEM 65

A two-lane arterial highway has a design speed of 55 mph and a design ADT of 5030 vpd. The roadway has 12-foot lanes and 8-foot shoulders, and is located on an embankment with of slopes 5H:1V. A utility company plans to install poles along the right side of the highway for a new electric transmission line in an area where the roadway curves to left on a 1150 feet radius curve. The utility should place the poles beyond what range of required clear-zone distance to avoid the need to protect them?

SOLUTION 65

The roadway is on an embankment (or on fill slope), therefore refer to the columns under the heading "FORESLOPES". The actual foreslope is given as 5H:1V, the design speed as 55 mph, and the design ADT as 5030 vpd.

Therefore the *basic* range of required clear-zone distance is found at the intersection of the row marked "55 mph/ 1500-6000 ADT", and the column marked, "FORSLOPE/ 1V:5H TO 1V:4H", as 24 – 30 ft.

Since the proposed utility poles are located on the outside of a horizontal curve (right side of a curve to the left) a curve correction factor must be applied to the *basic* range of required clear-zone distance. Therefore refer to the following table.

Table 3-2. Horizontal Curve Adjustment Factor

Radius, m [ft]	Design Speed km/h [mph]					
	60 [40]	70 [45]	80 [50]	90 [55]	100 [65]	110 [70]
900 [2,950]	1.1	1.1	1.1	1.2	1.2	1.2
700 [2,300]	1.1	1.1	1.2	1.2	1.2	1.3
600 [1,970]	1.1	1.2	1.2	1.2	1.3	1.4
500 [1,640]	1.1	1.2	1.2	1.3	1.3	1.4
450 [1,475]	1.2	1.2	1.3	1.3	1.4	1.5
400 [1,315]	1.2	1.2	1.3	1.3	1.4	—
350 [1,150]	1.2	1.2	1.3	1.4	1.5	—
300 [985]	1.2	1.3	1.4	1.5	1.5	—
250 [820]	1.3	1.3	1.4	1.5	—	—
200 [660]	1.3	1.4	1.5	—	—	—
150 [495]	1.4	1.5	—	—	—	—
100 [330]	1.5	—	—	—	—	—

For 1150 feet radius curve and 55 mph design speed, the appropriate correction factor is **1.4**. Multiply the *basic* range of required clear-zone distance by the correction factor.

The adjusted range is **33.6 – 42 ft**.

14.2 Impact Attenuator

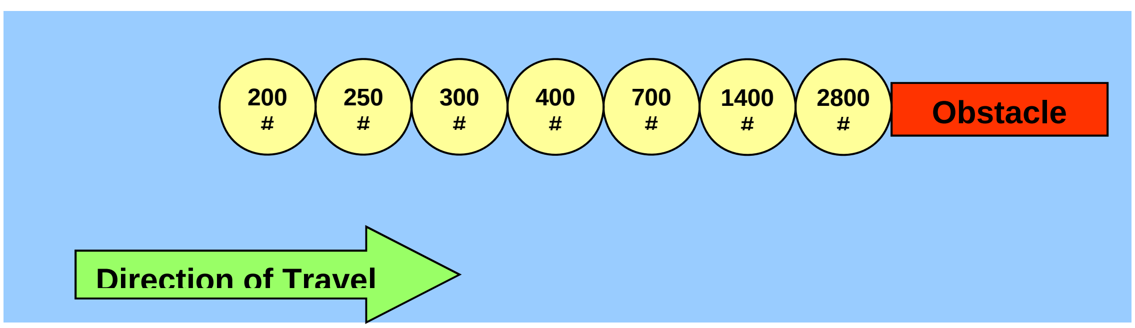
Also called Crash Cushions. These are protective devices that prevent errant vehicles from impacting fixed objects.

For more information on the principle of conservation of momentum as applied to sand-filled barrel inertial impact attenuators, see the AASHTO Roadside Design Guide, 4th Edition, 2011, page 8-36 & 8-38

PROBLEM 66

A 3000 pound test vehicle is traveling at 45 mph when it suddenly leaves the test track and heads directly toward an obstacle that is protected by the sand-filled barrel impact attenuator system shown below. Each barrel is 3'-0" in diameter.

a) How fast is the vehicle traveling the moment it hits the fourth barrel?
b) How many G's are experienced by the occupants the moment the vehicle hits the fourth barrel?
c) If the vehicle hits the first barrel at time t = 0.0 seconds, at what time does the vehicle hit the fifth barrel?



SOLUTION 66

Use the following equations (1) through (4) below to calculate:

Use all four equations once for each barrel. Enter the results in the following table.

$$Equation\ 1: V_1 = \frac{M_V V_o}{M_V + M_1}$$

$M_V = mass\ of\ vehicle\ (lb)$
$V_O = original\ vehicle\ velocity\ (ft/sec)$
$M_1 = mass\ of\ sand\ barrel\ (lb)$
$V_1 = vehicle\ velocity\ after\ impact\ (ft/sec)$

$$Equation\ 2: a = \frac{V_O^2 - V_1^2}{2D}$$

$a = deceleration\ rate$ (ft/sec^2)
$D = decelartion\ distance\ (barrel\ diameter\ in\ ft)$

$$Equation\ 3: G = \frac{a}{g}$$

g = acceleration of gravity (32.2 ft/sec^2)
$G = deceleration\ force$

$$Equation\ 4: t = \frac{V_O - V_1}{a}$$

$t = elapsed\ time\ (sec)$

	Calculation Results Table				
M_1 (lb)	V_O (ft/s)	V_1 (ft/s)	$a(ft/sec^2)$	G	$t(s)$
200	66.15	62.02	88.23	2.74	0.047
250	62.02	57.25	94.82	2.94	0.050
300	57.25	52.04	94.90	**2.95**	0.055
400	**52.04**	45.92	99.92	3.10	0.061

a) How fast is the vehicle traveling the moment it hits the fourth barrel? **52 fps**
b) How many G's are experienced by the occupants as the vehicle hits the fourth barrel? **2.95**
c) If the vehicle hits the first barrel at time t = 0.000 seconds, at what time does it hit the fifth barrel? **0.213 seconds** = (0.047 + 0.050 +0.055 + 0.061)

PROBLEM 67

An impact attenuator must be placed at an elevated gore which will provide an acceptable deceleration level for a 5,000 lb vehicle travelling at 50 mph. 3 ft in diameter barrel impact attenuators are available to place at the elevated gore. If each barrel can absorb 12.5 kip-ft coefficient of energy, what is the minimum number of barrels required to bring the deceleration of the vehicle to acceptable levels?

SOLUTION 67

Step 1 – Calculate moving vehicle's kinetic energy using equation 13.13(b) on Page 13-3 in CERM 14

$$E_{Kinetic} = \frac{mv^2}{2g_c}$$

m = Mass (Weight) of the vehicle in lbs = 5,000 lb
v= Vehicle speed in ft/s = 50 * 1.47 = 73.5 ft/sec
g_c= Gravitational Constant = 32.2 ft/sec^2

$$E_{Kinetic} = \frac{5{,}000 * \ 73.5^2}{2 * 32.2} = 419{,}429 \text{ ft} - \text{lbs} = 419 \text{ kip} - \text{ft}$$

Step 2 – Compute minimum number of barrels required

If each barrel can absorb 12.5 kip-ft of energy,

Minimum number of barrels required = 419/12.5 = 33.5 = 34 barrels

15. Pavement Design

- Estimate Design Traffic using AASHTO Guidelines
- Design Pavement

15.1 Design Traffic Volumes

- Estimate total volumes up to Design Year using growth factors and the existing volumes
- Convert into ESALs – Equivalent Single-Axle Loads. To simplify pavement design calculations, the mixed stream of traffic must be converted into an equivalent number of 18-kip (18,000 pound) single-axle load (ESALs). This conversion eliminates the problem of "adding apples and oranges" and is similar to how trucks, buses, and recreational vehicles are converted to passenger car equivalents for purposes of capacity analysis.

ESAL Factors

Vehicle Types	ESAL Factors	Vehicle Types	ESAL Factors
Passenger Cars	0.0008	Buses	0.6806
Panel/pickup Trucks	0.0122	5 Axle Double Trailers	2.3187
Other 2-Axle 4-Tire Trucks	0.0052	6+ Axle Double Trailers	2.3187
2-Axle/6-Tire Trucks	0.1890	All Double Trailer Combos	2.3187
3 or More Axle Trucks	0.1303		
All Single Unit Trucks	0.1303		
3-Axle Tractor Semi-Trailers	0.8646	3-Axle Tractor Truck-Trailers	0.0152
4-Axle Tractor Semi-Trailers	0.6560	4-Axle Tractor Truck-Trailers	0.0152
5+Axle Tractor Semi-Trailers	2.3719	5+Axle Tractor Truck-Trailers	0.5317
All Tractor Semi-Trailers	2.3719	All Tractor Truck- Combos	0.5317

Source: *AASHTO Guide for Design of Pavement Structures*, 1993 by the American Association of State Highway and Transportation officials, Washington, D.C.

- Estimate ESALs for the design lane using CERM equation 76.26 and table 76.7 on page 76-17.

PROBLEM 68
A four-lane arterial has an AADT of 12,000 with a directional distribution factor of 0.53 and the following truck classifications and percentages. Assuming an average growth rate of 2.0% over a 20-year design period, determine the ESALs for the design lane.

Truck Classification	Panel/pick-up trucks	2-axle, 6-tire	3+ axles Trucks	3-axle Semis	4-axle Semis	5-axle Semis
Percent of AADT	11%	6%	4%	2%	2%	15%

SOLUTION 68
The recorded AADT consists of a mixed stream of vehicles, axle weights and configurations.

Procedure. Solving this problem involves the following steps.
1. Determine the Growth Factor for the design period.
2. Create a table for all input data and calculations.
3. Calculate the Design Traffic by truck classification.
4. Obtain the appropriate ESAL factors from AASHTO tables.
5. Calculate the Total Design ESALs.
6. Determine the directional split and the total ESALs in the design lane.

1. **Growth Factor.** Traffic increases at a constant rate of 2.0% per year. To find the total number of vehicles that will use the road over a 20-year design period, find the associated growth factor, GF, as follows:

$$GF=\left[\frac{(1+r)^{n}-1}{r}\right]=\left[\frac{(1+0.02)^{20}-1}{0.02}\right]=\left[\frac{1.4859-1}{0.02}\right]=\frac{0.4859}{0.02}=24.3$$

where: r = growth rate

n = design period (years)

GF = growth factor - enter in Column (5)

2. **Create Table.** Organize the data provided as shown in Table below, columns (1), (2), and (3). Column (4) lists the total number of days per year: 365.

Truck Classification	AADT	Fraction of AADT	Days per Year	Growth Factor	Design Traffic	ESAL factors *	ESALs
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Panel/pickup	12,000	0.11	365	24.3	11,707,740	0.0122	142,834
2-axle, 6-tire	12,000	0.06	365	24.3	6,386,040	0.1890	1,206,962
3+ axles Trucks	12,000	0.04	365	24.3	4,257,360	0.1303	554,734
3-axle Semis	12,000	0.02	365	24.3	2,128,680	0.8646	1,840,457
4-axle Semis	12,000	0.02	365	24.3	2,128,680	0.6560	1,396,414
5-axle Semis	12,000	0.15	365	24.3	15.965.100	2.3719	37,867,621
		0.40		**Total Design ESALs:**			**43,009,021**

SOURCE: CERM Example 76.5

3. **Design Traffic.** Multiply columns (2), (3), (4) and (5), to find the total (cumulative) number of each truck classification that will use the road over the design period. Enter in column (6).

4. **ESAL Factors**. Obtain the appropriate ESAL factors for the given truck classifications from AASHTO table of ESAL Factors. Enter in column (7).

5. **Total Design ESALs**. Multiply columns (6) and (7), to find the total ESALs per truck classification. Enter in column (8). The sum of column (8) is the Total Design ESALs: 43,009,021.

6. **ESALs in Peak Direction and Design Lane.** Using CERM equation 76.26, $w_{18} = D_D D_L \overline{w}_{18}$

D_D = 0.53 (given)
D_L = 0.90 average of 0.80-1.0 (CERM Table 76.7)
$\overline{w}_{18}$ = 43,009,021
$w_{18} = D_D D_L \overline{w}_{18}; = 0.53 \times 0.90 \times 43{,}009{,}021 = 20{,}515{,}303 \text{ ESALs}$

15.2 Flexible Pavement Design

- Asphalt surface
- Using AASHTO Guidelines
  - Estimate Design Traffic
  - Determine the strength of the pavement layer and the underlying soil
  - Compute layer-thickness

15.2.1 Layer Strengths

- Goal is to Design the Structural Number (SN) for the given traffic, roadway, and soil characteristics

PROBLEM 69
Given the following information, design the structural number for the flexible pavement:

- Estimated future traffic, $w_{18} = 5 * 10^6$
- Reliability, R = 95%
- Standard Deviation, $S_0 = 0.35$
- Effective modulus of the roadbed material, M_R = 5,000 psi
- Design serviceability loss, ∇PSI = 1.9

SOLUTION 69
See CERM Page 76-22

15.2.2 Pavement Thickness

- Goal is to estimate the pavement thickness for the given SN and pavement material characteristics
- Use CERM equation 76.33 on page 76-23

PROBLEM 70
Determine the thickness of flexible pavement layer 2 given the following:

FACTS:
Total Structural Number required = 7.0
Total number of layers = 2
Layer 1 consists of asphalt concrete with a strength coefficient of 0.46
Layer 1 thickness = 6 inches
Layer 2 consists of granular base with a strength coefficient of 0.15 and a drainage coefficient of 1.0

SOLUTION 70

Using CERM equation 76.33, $SN = D_1 a_1 + D_2 a_2 m_2$

$$D_2 = \frac{SN - D_1 a_1}{a_2 m_2} = \frac{7.0 - 6 \times 0.46}{0.15 \times 1.0} = 28.3"$$

15.3 Rigid Pavement Design

- Similar to Flexible Pavement Design
  - Estimate ESALs
  - Determine pavement thickness using graphs on CERM pages 77-8 and 77-9 for the given parameters (no need to calculate SN as in flexible pavement)

PROBLEM 71

Given the following information, design the slab thickness for the rigid pavement:

- Estimated future traffic, $w_{18} = 5.1 * 10^6$
- Reliability, R = 95%; Standard Deviation, $S_0 = 0.29$
- Effective modulus of subgrade reaction, k = 72 lbf/in^3
- Concrete elastic modulus, $E_c = 5 \times 10^6$ lbf/in^2
- Mean concrete modulus of rupture, $S_c' = 650$ lbf/in^2
- Load transfer coefficient, J = 3.2; Drainage coefficient, $C_d = 1.0$
- Design serviceability loss, ∇PSI = 1.9

SOLUTION 71

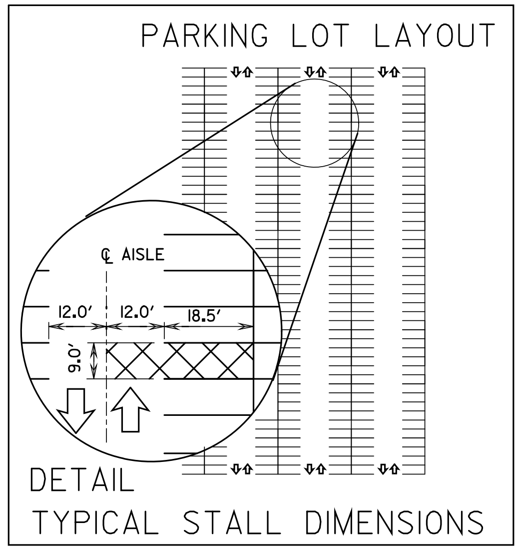
See CERM pages 77-8 and 77-9

16. Parking Lot Spaces

See CERM Page 73-21

PROBLEM 72

Design criteria for a proposed paved parking lot requires that parking stalls be 9.0 ft wide by 18.5 ft deep, and oriented perpendicular (90^0) to two-way aisles that are 24 ft wide. Additional paved areas equal to at least 2% of the area of the aisles and stalls must be provided for access to the aisles. Also, a vegetated area equal to at least 10% of the total paved area must be provided for grass and landscaping.



Based on the above criteria,

a) what is the approximate number of stalls that can fit on a 4-acre site?

b) What is the minimum number of accessible parking spaces to be designated for this parking lot?

SOLUTION 72

Part a)

Determine the area required for one parking stall, including the associated portion of the adjacent aisle.

$$\text{Area}_{\text{stall}} = \text{Width}_{\text{stall}}(L_{\text{stall}} + 1/2\ W_{\text{aisle}}) = 9.0[18.5 + 1/2(24)] = 274.5\ \text{ft}^2$$

Determine the total paved area required per parking stall.

$$\text{Paved Area}_{\text{stall}} = 274.5\ \text{ft}^2 \times 1.02 = 279.99\ \text{ft}^2$$

Determine the total site area (paved + vegetation) required per parking stall.

$$\text{Total Site Area}_{\text{stall}} = 279.99\ \text{ft}^2 \times 1.10 = 307.99\ \text{ft}^2$$

Determine the number of parking stalls that will fit on a 4-acre site.

$$\text{no. of stalls} = \left(\frac{4\ \text{acres} \times 43{,}560\ \text{ft}^2/\text{ac}}{307.99\ \text{ft}^2/\text{stall}}\right) = 565.73\ \text{stalls}\ (\underline{565\ \text{stalls}})$$

Part b)

See CERM Table 73.17

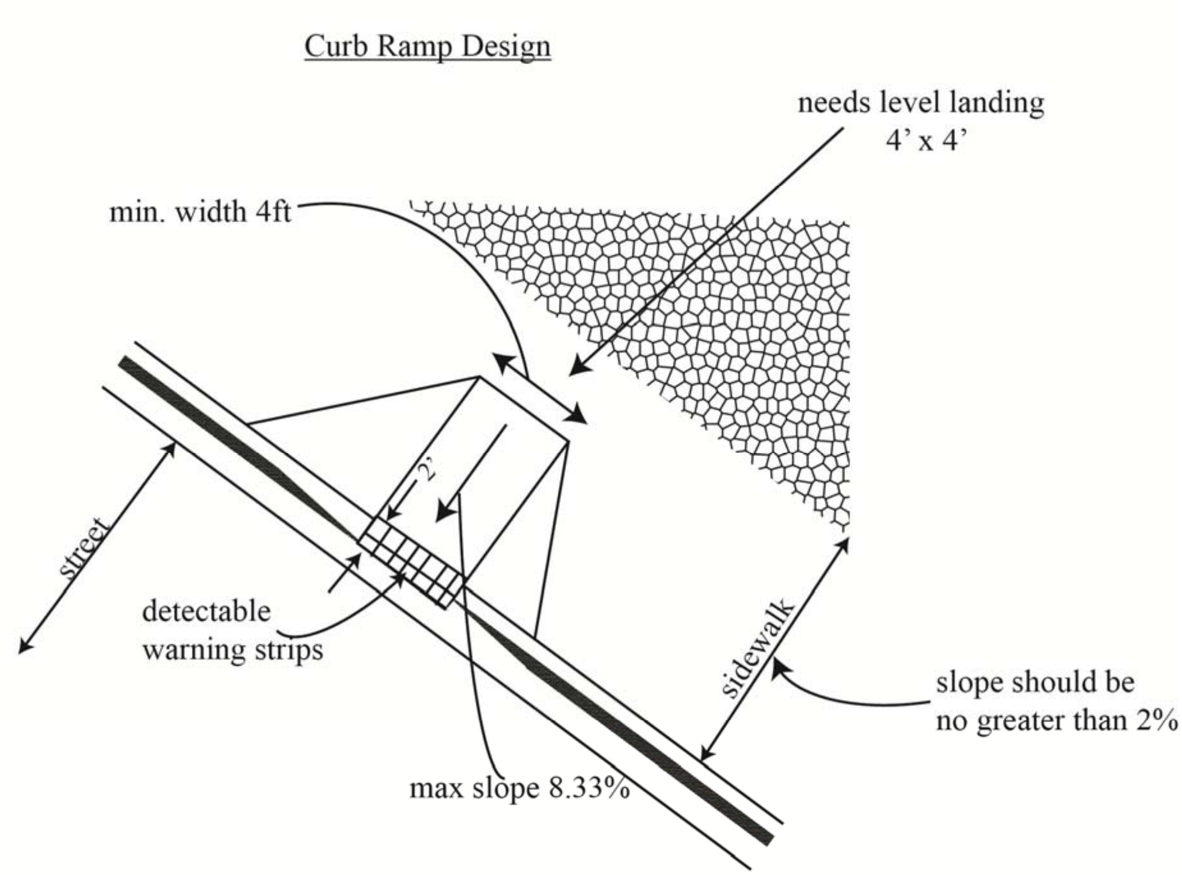
For 501 to 1000 parking spaces, minimum 2% should be designated as Accessible Parking Spaces:

2% of 565 = 12 spaces

17. Pedestrian Facilities

Use "Policy on Geometric Design of Highways and Streets" 2011 by AASHTO (Green Book)

See Pages 4-61 through 4-65 in Green Book



18. Turning Radii of Design Vehicles

Use "Policy on Geometric Design of Highways and Streets" 2011 by AASHTO (Green Book)

See Section 2.1.2, Table 2-2b, and Figures 2-1 thru 2-23 in Green Book

19. Queuing Analysis

Types:

- M/M/1 – Arrival with Poisson/Departure with Poisson/Through One Channel
- D/D/1- Arrival with Uniform Deterministic/Departure with Uniform Deterministic/Through One Channel

Scenario 1 – Arrival Rate $\leq$ Departure Rate (Based on M/M/1)

- Arrival Rate - λ
- Departure Rate or Service Rate - μ
- Utilization factor, $\rho = \lambda/\mu$
- Average time per vehicle is queue = $\lambda/(\mu(\mu-\lambda))$
- Average time in service = $1/\mu$
- Average time per vehicle in system = Average time in queue + Average time in service = $1/(\mu-\lambda)$
- Average Queue length (number of vehicles waiting in line to be serviced) = $\rho^2/(1-\rho)$
- Average number of vehicles in system (number of being served + number in queue) = $\rho/(1-\rho)$
- Probability of finding 'n' vehicles in the system, $\rho(n) = \rho^n(1 - \rho)$

Scenario 2 – Arrival Rate > Departure Rate (Based on D/D/1)

- Possibility of having infinite queue length. Therefore, this scenario is typically for shorter durations.
- Total vehicle delay = Total area
- Average delay = Total vehicle delay/total number of vehicles arrived

PROBLEM 73

The arrival rate of vehicles entering a parking lot is 150 vehicles per hour. Access to the lot is accommodated by way of a single, gated driveway. The service rate of the gate is 200 vph. Both arrival and service rates are with Poisson distribution. Find:

a) The utilization factor for the above parking lot entry system
b) The Average Queue Length
c) Average number of vehicles in system

SOLUTION 73

a. $\rho = \lambda/\mu = 150/200 =$ **0.75**

b. Avg Queue length: $E(m) = \rho^2/(1-\rho)$;
$E(m) = (0.75)^2/(1-0.75) =$ **2.25 vehicles**

c. Average number of vehicles in system: $E(n) = \rho/(1-\rho)$;
$E(n) = (0.75)/(1-0.75) = 3$ **vehicles**

PROBLEM 74

A multi-vehicle crash occurs on the eastbound lanes of a busy urban arterial at **4:30 p.m.** The incident blocks both eastbound lanes until **4:50 p.m.** when emergency crews manage to clear one of the lanes and eastbound traffic flow is restored at a rate of **1680 veh/h**. At **5:10 p.m.** all disabled vehicles and debris are cleared, and the eastbound lanes are restored to full capacity (**3600 veh/h**).

Assume there is no diversion of eastbound traffic off of the freeway as a result of the crash, that under normal p.m. traffic conditions the eastbound lanes have a constant flow of **2700 veh/h,** and that traffic continues to flow eastbound at the normal rate until forced to stop for the queue. Determine the time of queue disappears assuming D/D/1 queuing.

SOLUTION 74

Let μ be the full capacity departure rate and μ_r be the restrictive partial-capacity departure rate. Putting arrival and departure rates in common units of vehicles per minute,

μ = 3600 veh/h / 60 min/h = 60 veh/min; full capacity
μ_r = 1680 veh/h / 60 min/h = 28 veh/min; partial-capacity
λ = 2700 veh/h / 60 min/h = 45 veh/min; arrival rate

The arrival rate is constant over the entire period, and the total number of arriving vehicles is equal to λt, where t is the number of minutes after 4:30 p.m.

The total number of departing vehicles is computed based on the following:

0	$t \leq 20$ min
$\mu_r(t-20)$	for 20 min $< t \leq 40$ min
$560 + \mu(t-40)$	for $t > 40$ min

Note that the value of 560 in the departure rate function for t > 40 is based on the preceding departure rate function [28(40 – 20)]. These arrival and departure rates are graphed below.

Note that for D/D/1 queuing, the queue dissipates at the intersection of the arrival and departure curves, which can be determines mathematically as

$$\lambda t = 560 + \mu(t-40)$$
$$45t = 560 + 60(t-40)$$

where t = <u>122.67 min</u> (6:32:67 p.m.)

Queue is cleared after 122.67 minutes from the time of the crash or cleared at 6:32:67 p.m.