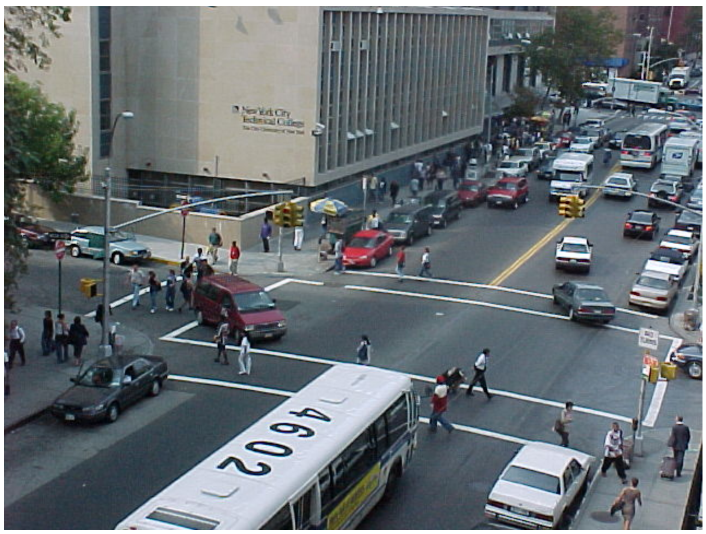
TRAFFIC ENGINEERING

5^{TH} Edition



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SOLUTIONS MANUAL

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Solutions to Problems in Chapter 2

Transportation Modes and Characteristics

Problem 2-1
The capacity of a street or highway is affected by a) the physical design of the roadway – such features as the number of lanes, free-flow speed, and geometric design, b) the traffic composition – particularly the presence of trucks and local buses, and c) the control environment – such features as lane use controls, signalization, curb lane controls, etc.

Problem 2-2
The capacity of a rapid transit line is affected by: the number of tracks, the person-capacity of each rail car, the length of trains, and the minimum headways at which trains can operate. The latter is limited by either the control system or station dwell times.

Problem 2-3
The key element here is that trains may operate 1.8 minutes apart. In this case, the dwell time controls this limit, not the train control system, which would allow closer operation. Thus, one track can accommodate 60/1.8 = 33.3 (say 33) trains/h.

Each train has 10 cars, each of which accommodates a total of 50+80 = 130 passengers. The capacity of a single track is, therefore:

$$33*10*130 = 42{,}900 \text{ people/h}$$

Problem 2-4
From Table 2-5 of the text, a freeway with a free-flow speed of 55 mi/h has a vehicle-capacity of 2,250 passenger cars/h.

Traffic contains 10% trucks and 2% express buses, each of which displaces 2.0 passenger cars from the traffic stream. At capacity, there are:

2,250*0.10 = 225 trucks
2,250*0.02 = 45 express buses

Each of these displaces 2.0 passenger cars from the traffic stream. Thus, the 225+45 = 270 heavy vehicles displace 2*270 = 540 passenger cars from the traffic stream. Thus, the number of passenger cars at capacity is:

2,250 – 540 = 1,710 passenger cars

Using the vehicle occupancies given in the problem statement, the person-capacity of one lane is:

$$(1710*1.5)+(225*1.0)+(45*50) = 5{,}040 \text{ persons/h}$$

As there are 3 lanes in each direction, the capacity of each direction is 3*5040 = 15,120 people/h.

Problem 2-5

A travel demand of 30,000 persons per hour is virtually impossible to serve entirely with highway facilities. Even in the best case of a freeway with a 70-mi/h free-flow speed, and an assumed occupancy of 1.5 persons/car, a lane can carry only 3,600 people/h (Table 2-5). That dictates a need for 30,000/3,600 = 8.33 fully-dedicated freeway lanes to serve this demand. While this might be technically feasible if the area were basically vacant land with a new high-density trip generator being built, it would be intractable in most existing development settings.

That leaves various public transit options (Table 2-6). Given the observed capacities, it is doubtful that such a demand could be handled by bus transit (either on the street or on a private right-of-way) or light rail. A rapid transit line with one track in each direction would be able to handle the demand.

A lot depends on what type of development is spurring the demand. If it is a stadium or entertainment complex that generates high-intensity demand for short periods of time, the solution may be different from a case of a regional shopping mall, where trips are more distributed over time.

It is likely that some mix of modes would be needed. Rail transit is expensive, and any new service would have to be linked into a larger rapid transit network to be useful. Auto access is generally preferred by users (except for the traffic it generates), but involves the need to provide huge numbers of parking places within walking distance of the desired destination. A stadium could rely fairly heavily on transit, with heavy rail, light rail, and bus options viable. Some highway access and parking would also be needed. A regional shopping center would have to cater more to autos, as most people would prefer not to haul their purchases on transit.

Solutions to Problems in Chapter 3

Speed, Travel Time, and Delay Studies

Problem 3-1
The reaction distance is given by Equation 3-1:

$$d_r = 1.47\,S\,t$$

For a speed of 70 mi/h, the result is:

$$d_r = 1.47 * 70 * 3.5 = 360.2 \ ft$$

Other values for the range of speeds specified are shown in Table 1. Figure 1 plots these values.

Table 1: Reaction Distance vs. Speed

Speed	Distance
30	154.4
35	180.1
40	205.8
45	231.5
50	257.3
55	283.0
60	308.7
65	334.4
70	360.2

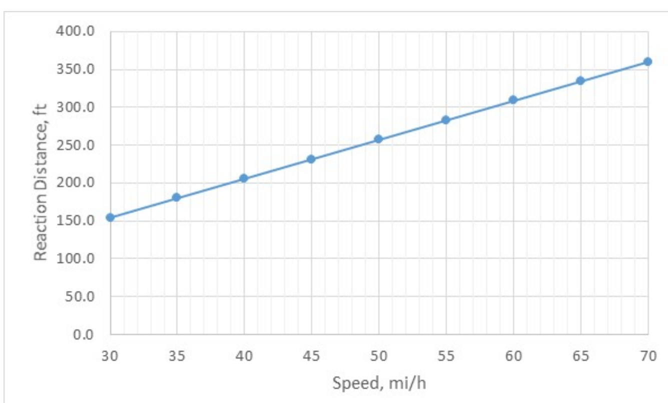


Figure 1: Reaction Distance vs. Speed

Problem 3-2

This problem involves several considerations. At the point when the driver notices the truck, the vehicle is 350 ft away from a collision. To stop, the driver must go through the reaction distance and then the braking distance. The two will be considered separately.

Reaction Distance

Reaction distance is given by Equation 3-1, and is dependent upon the reaction time, which, for this problem, will be varied from 0.50 s to 5.00 s. A sample solution for 0.50 s is shown, with all results in Table 3.

$$d_r = 1.47\,S\,t = 1.47 * 65 * 0.50 = 47.8 \ ft$$

Table 3: Reactions Distances for Problem 3-2

Speed (mi/h)	Reaction Time (s)	Reaction Distance (ft)
65	0.50	47.8
65	1.00	95.6
65	1.50	143.3
65	2.00	191.1
65	2.50	238.9
65	3.00	286.7
65	3.50	334.4
65	4.00	382.2
65	4.50	430.0
65	5.00	477.8

For any result > 350 ft, the driver will not even get his/her foot on the brake before colliding with the truck. Thus, for all reaction times, t ≥ 4.0 s, the collision speed will be 65 mi/h.

Braking Distance

For all reaction times < 4.0 s, the driver will engage the brake before hitting the truck, and therefore, will at least decelerate somewhat before a collision. How much deceleration will take place depends upon how much braking distance is left when the brake is engaged. In each case, this would be 350 ft – the reaction distance, d_r. Once the braking distance available is determined, the braking distance formula of Equation 3-5 is used:

$$d_b = \frac{S_i^2 - S_f^2}{30\,(F \pm G)}$$

The braking distance will be determined as indicated. For example, for a reaction time of 2.0 s, the reaction distance (from Table 3) is 191.1 ft. The available braking distance is then 350.0 – 191.1 = 158.9 ft. The initial speed (S_i) is 65.0 mi/h in all cases. The grade is given as level (G = 0.00), and the friction factor is found from the deceleration rate as:

$$F = \frac{a}{g} = \frac{10}{32.2} = 0.311$$

The final speed (S_f) is the unknown. Then for the example with a 2.0 s reaction time:

$$d_r = 158.9 = \frac{65^2 - S_f^2}{30(0.311 + 0.000)}$$

$$S_f^2 = 65^2 - (158.9 * 30 * 0.311) = 2{,}739.5$$

$$S_f = 52.4\ mi/h$$

Table 4 summarizes the results for all reaction times.

Table 4: Collision Speed vs. Reaction Time for Problem 3-2

Reaction Time (s)	Braking Distance (ft)	Collision Speed (mi/h)
0.5	302.2	37.5
1.0	254.5	43.0
1.5	206.7	47.9
2.0	158.9	52.4
2.5	111.1	56.5
3.0	63.4	60.3
3.5	15.6	63.9
4.0	NA	65.0
4.5	NA	65.0
5.0	NA	65.0

Figure 2 shows a plot of these results. Note that in no case is the driver able to stop the vehicle before colliding with the truck.

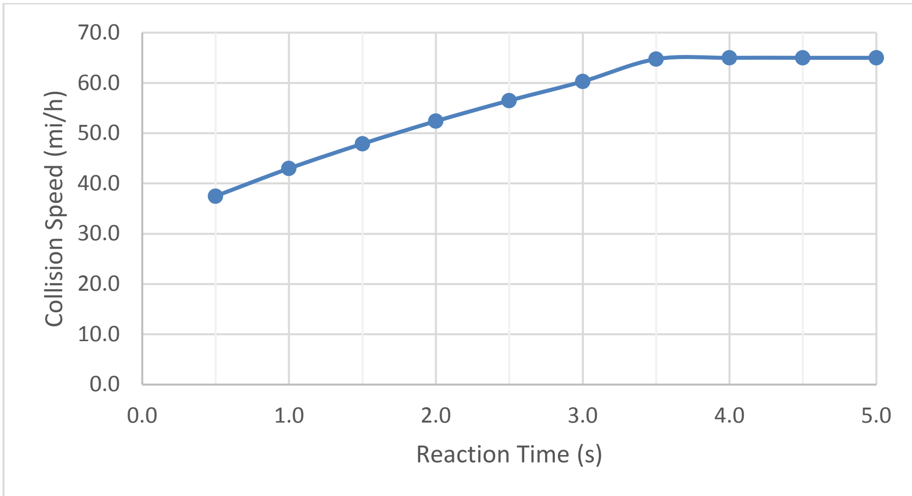
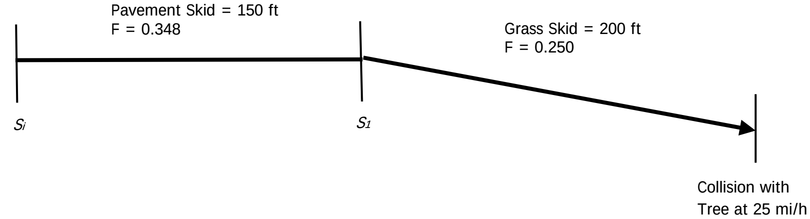


Figure 2: Reaction Time vs. Collision Speed, Problem 3-2

Problem 3-3

In this case, we are dealing with measured skid marks at an accident location. Because skid marks only occur when the brakes are engaged, the reaction time and reaction distance play no role in this solution.

The sketch below helps in the understanding of the solution:



The only know speed is at the collision point (at the end of the grass skid). The collision speed is 25 mi/h. Using the grass skid, we can work backwards to find the initial speed at the beginning of the grass skid (S_1). This is also the final speed at the end of the pavement skid. Working backwards again, we can find the initial speed (S_i) at the beginning of the pavement skid.

Both solutions use the braking formula of Equation 3-5:

$$d_b = \frac{S_i^2 - S_f^2}{30(F \pm G)}$$

$$d_{b,GRASS} = 200 = \frac{S_1^2 - 25^2}{30(0.250 + 0.03)}$$

$$S_1^2 = (200 * 30 * 0.280) + 25^2 = 2{,}305$$

$$S_1 = 48.0\ mi/h$$

$$d_{b,PAVE} = 150 = \frac{S_i^2 - 48.0^2}{30(0.348 + 0.03)}$$

$$S_i^2 = (150 * 30 * 0.378) + 2{,}305 = 4{,}006$$

$$S_i = 63.3\ mi/h$$

In an accident investigation, this result would be compared to the speed limit to determine whether excessive speed contributed to the accident.

Problem 3-4

This problem involves a reaction distance and a braking distance, as drivers must see a sign and reduce their speed to navigate a hazard. Level terrain is assumed, and standard values for *t* (2.5 s) *F* (0.348) and *a* (10.0 ft/s^2) are used. The full distance to respond is the sum of Equation 3-1 for reaction distance and Equation 3-5 or 3-6 for braking:

$$d = 1.47\,S_i\,t + \frac{S_i^2 - S_f^2}{30(0.348 \pm G)} = (1.47 * 60 * 2.5) + \frac{60^2 - 40^2}{30 * 0.348} = 220.5 + 191.6 = 412.1\ ft$$

The sign must *be seen* a total of 412.1 ft from the hazard. Since the sign can be read from 200 ft, it could be placed as close as 412.1-200.0 = 212.1 ft from the hazard. Other considerations, however, would also enter a final decision on the placement of the sign.

Problem 3-5

The *yellow* interval of a traffic signal is designed to let any vehicle that cannot safely stop before entering the intersection safely enter the intersection at the ambient speed, which is generally taken to be an 85th percentile speed. First, the safe stopping distance must be found for a vehicle traveling at 40 mi/h on a 0.02 downgrade, using Equation 3-10:

$$d_S = 1.47\,S\,t + \frac{S^2}{30(0.348 \pm G)} = (1.47 * 40 * 1.0) + \frac{40^2}{30(0.348 - 0.02)} = 58.5 + 162.6 = 221.1\ ft$$

As the vehicle is traveling at a speed of 40 mi/h, the *yellow* must be long enough to allow the vehicle to traverse 221.1 ft at 40 mi/h, or:

$$y = \frac{221.1}{1.47 * 40} = 3.76\ s$$

Problem 3-6

The safe stopping distance is computed using Equation 3-10:

$$d_S = 1.47\,S\,t + \frac{S^2}{30(0.348 \pm G)} = (1.47 * 80 * 2.5) + \frac{80^2}{30(0.348 - 0.02)} = 294.0 + 650.4 = 944.4\ \ ft$$

Problem 3-7

The minimum radius of curvature is given by Equation 3-3:

$$R = \frac{S^2}{15(e + f)} = \frac{70^2}{15(0.06 + 0.10)} = 2{,}041.7\ \ ft$$

Solutions to Problems in Chapter 4

Communicating With Drivers: Traffic Control Devices

Problem 4-1

A *standard* in the MUTCD is a mandatory condition, and is accompanied by the words "shall" or "shall not." Standards must be followed, and failure to do so leaves the agency in charge with potential legal liability for accidents.

A *guideline* in the MUTCD is strongly suggested advice based upon national consensus in the profession. While not legally binding, any deviation should be documented by an engineering study that is kept on file. Legal liability may still exist, especially where no documentation of the deviation is available. A guideline is accompanied by the words "should" or "should not."

An *option* in the MUTCD is just that – an option. It presents information that may be implemented or not based upon the local judgment of the relevant traffic agency. No legal liability is implied.

Support in the MUTCD is simply additional information for the manual user.

Problem 4-2

Human eyesight can identify color, then shape or pattern, and finally, specific text. Because color and shape are discernible to most road users at great distances, they are used to code types of information, and to draw the attention of road users requiring this type of information. Thus, the STOP sign has a unique color, shape, and legend. The word "STOP" could easily be omitted, and drivers would still know the meaning of the red octagon. Guide signs are also color- and shape-coded. Directional information is on rectangular signs (long dimension horizontal) with a green background. Services information is on similar-shaped signs, but with a blue background. Cultural or historic directional information is on rectangular signs too, but with a brown background. All warning signs are diamond-shaped with a yellow background.

Problem 4-3

Overuse of warning signs is particularly dangerous. If drivers begin to suspect that warning signs are not warning about things that are truly an imminent threat, they may tend to ignore them – which would be a major problem in the case of a real imminent threat. They should be used to bring drivers' attention to an upcoming hazard that they would not normally be able to discern in time to safely maneuver through it.

Regulatory signing should also be used only when needed to inform drivers about a regulation that they would otherwise be unaware of. Overuse again tends to cause drivers to not pay attention to them.

Guide signs are unique in that only a small percentage of drivers actually use them: familiar drivers by-and-large know where they are going and how to get there. For others, however, frequent guidance is a comfort, and avoids having confused drivers – who are, by definition – dangerous drivers.

Problem 4-4

Table 4-1 is consulted to determine appropriate posting locations.

a) For a STOP ahead sign, the advisory speed is an implied “0” mi/h. Using Condition B with a speed limit of 50 mi/h and an advisory speed of 0 mi/h, the sign would be placed 250 ft in advance of the location of the STOP sign.

b) For a curve ahead sign, Condition B is used with a speed limit of 45 mi/h and an advisory speed of 30 mi/h. The sign would be placed 100 ft from the curve, which is the minimum advance placement distance permitted. Signs are assumed to be visible for 250 ft away.

c) A merge ahead sign may be viewed as a Condition A maneuver. For a speed limit of 35 mi/h, the sign would be placed 565 ft from the merge point.

Problems 4-5 and 4-6

Both of these are local projects for students. They require field observation of the students. Instructors should check to see whether students are insured (by the University) for such activities.

Solutions to Problems in Chapter 5

Traffic Stream Characteristics

Problem 5-1

a) The flow rate is computed as:

$$v = \frac{3{,}600}{h_{av}} = \frac{3{,}600}{2.6} = 1{,}385\ veh/h$$

b) The density is computed as:

$$D = \frac{5{,}280}{d_{av}} = \frac{5{,}280}{235} = 22.47\ veh/mi$$

c) The average speed is computed as:

$$S = \frac{v}{D} = \frac{1{,}385}{22.47} = 61.6\ mi/h$$

Problem 5-2

The peak hour factor is defined as:

$$PHF = \frac{V}{v}$$

where:	V	=	peak hour volume, vehs/h, and
	v	=	peak rate of flow within the hour, vehs/h.

Therefore:

$$V = v * PHF$$

and:

a) V = 5,600*0.85 = 4,760 veh/h
b) V = 5,600*0.90 = 5,040 veh/h
c) V = 5,600*0.95 = 5,320 veh/h

Problem 5-3

The solution is best carried out using a spreadsheet. The spreadsheet that follows finds the desired results as:

a) The AADT is the total volume for the year (sum, Col. 4) divided by the total number of days in the year (sum, Col. 3.)

$$AADT = \frac{1,486,000}{365} = 4,071 \; vehs/day$$

b) The ADT for each month is the total volume for the month (Col. 4) divided by the total number of days in each month (Col. 3).

c) The AAWT is the total weekday volume for the year (sum, Col. 5) divided by the total number of weekdays in the year (sum, Col. 2).

$$AAWT = \frac{860,000}{260} = 3,308 \; vehs/day$$

d) The AWT for each month is the total weekday volume for the month (Col. 5) divided by the number of weekdays in the month (Col. 2).

Month	No. of Weekdays	Total Days	Total Monthly Vol	Total Weekday Vol	ADT	AWT
Jan	22	31	120,000	70,000	3,871	3,182
Feb	20	28	115,000	60,000	4,107	3,000
Mar	22	31	125,000	75,000	4,032	3,409
Apr	22	30	130,000	78,000	4,333	3,545
May	21	31	135,000	85,000	4,355	4,048
Jun	22	30	140,000	85,000	4,667	3,864
Jul	23	31	150,000	88,000	4,839	3,826
Aug	21	31	135,000	80,000	4,355	3,810
Sep	22	30	120,000	72,000	4,000	3,273
Oct	22	31	112,000	62,000	3,613	2,818
Nov	21	30	105,000	55,000	3,500	2,619
Dec	22	31	99,000	50,000	3,194	2,273
	260	**365**	**1,486,000**	**860,000**		

The information reveals two things about this facility:

1. Since the AADT > AAWT, and the monthly ADTs are generally larger than the monthly AWTs, this is likely a recreational route attracting mostly weekend travelers.

2. Traffic peaks in the summer months, for both AWT and ADT. It suggests that during the winter, many commuters may be away on vacation, and that the recreational region served mostly consists of summer activities.

Problem 5-4

Density is computed from occupancy measured at a detector as:

$$D = \frac{5{,}280 * 0}{L_v + L_d} = \frac{5{,}280 * 0.15}{6 + 20} = \frac{792}{26} = 30.5 \ \ veh/mi/\ln$$

Problem 5-5

A spreadsheet is used to determine the total hourly volume for each available hour of four consecutive 15-minute counts. This will identify the peak hour and the peak hour volume of parts a) and b), and permit the computations required in parts c) and d). The spreadsheet is shown below.

Time Period	Volume (vehs)	Cumulative Volume (vehs)
4:00-4:15	300	
4:15-4:30	**325**	
4:30-4:45	**340**	
4:45-5:00	**360**	1325
5:00-5:15	**330**	**1355**
5:15-5:30	310	1340
5:30-5:45	280	1280
5:45-6:00	240	1160

Then:

a) The peak hour occurs between 4:15 and 5:15 PM.

b) The peak hour volume is 1,355 vehs/h.

c) The peak rate of flow within the peak hour is 4*360 =1,440 vehs/h

d) The peak hour factor (PHF) is computed as:

$$PHF = \frac{V}{4 * V_{15}} = \frac{1{,}355}{4 * 360} = \frac{1{,}355}{1{,}440} = 0.941$$

Problem 5-6

The trick is that the volume must be divided into two lanes, or 1,800/2 = 900 veh/h/ln. Then, the density is computed as:

$$D = \frac{V}{S} = \frac{900}{40.0} = 22.5\, vehs/mi/\ln$$

Problem 5-7

An estimate of the directional design hour volume (DDHV) is found as:

$$DDHV = AADT * K * D$$

From Table 6-2 of the textbook, the range of K-factors applying to urban radial facilities is 0.07 – 0.12. The range of D-factors applying to urban radial facilities is 0.55 –0.60. Then:

$$DDHV_{LOW} = 150{,}000 * 0.07 * 0.55 = 5{,}775\ veh/h.$$
$$DDHV_{HIGH} = 150{,}000 * 0.12 * 0.60 = 10{,}800\ veh/h$$

Problem 5-8

The time mean speed (TMS) is the arithmetic average of individual vehicle speeds observed. Each speed is the distance (2,000 ft) divided by the travel time (s). This gives a result in ft/s, which should be converted to mi/h. The space mean speed (SMS) is the distance (2,000 ft) divided by the average of the individual travel times. The spreadsheet below helps illustrate these computations:

Veh	Length (ft)	t (s)	S (ft/s)	S (mi/h)
1	2,000	40.50	49.38	33.59
2	2,000	44.20	45.25	30.78
3	2,000	41.70	47.96	32.63
4	2,000	47.30	42.28	28.76
5	2,000	46.50	43.01	29.26
6	2,000	41.90	47.73	32.47
7	2,000	43.00	46.51	31.64
8	2,000	47.00	42.55	28.95
9	2,000	42.60	46.95	31.94
10	2,000	43.30	46.19	31.42
	SUM	438.00	457.82	311.44
	AVG	43.8	45.8	31.1

Then:

$$TMS = \frac{457.82}{10} = 45.8 \ ft/s$$
$$TMS = \frac{45.8}{1.47} = 31.2 \ mi/h$$

and:

$$SMS = \frac{2{,}000}{43.8} = 45.7 \ ft/s$$
$$SMS = \frac{45.7}{1.47} = 31.1 \ mi/h$$

<u>Problem 5-9</u>
The peak flow rate on the freeway lane is:

$$v = \frac{V}{PHF} = \frac{1200}{0.87} = 1{,}379 \ veh/h/\ln$$

<u>Problem 5-10</u>
The density on the freeway lane is found as:

$$v = S * D$$
$$D = {}^{v}\!/_{S} = {}^{1300}\!/_{35} = 37.1 \ veh/mi/\ln$$

<u>Problem 5-11</u>

a) The free-flow speed is 71.2 mi/h, the speed when density is “0.”
   The jam density is 122 pc/mi/ln, the density when speed is “0.”

b) To derive the speed-flow curve, substitute D=v/S in the equation:

$$S = 71.2\left(1 - \frac{D}{122}\right) = 71.2\left(1 - \frac{v/S}{122}\right) = 71.2 - \frac{71.2v}{122S}$$
$$\frac{71.2v}{122S} = 71.2 - S$$
$$71.2v = (122 * 71.2)S - 122S^2$$
$$v = 122S - 1.713S^2$$

To derive the flow-density curve, substitute S=v/D in the equation:

$$S = 71.2\left(1 - \frac{D}{122}\right) = \frac{v}{D}$$

$$\frac{v}{D} = 71.2 - \frac{71.2D}{122}$$

$$v = 71.2D - 0.584D^2$$

c) Capacity occurs when both the speed-flow and flow-density curves are at their peak, or when the first derivative of each is 0.0:

$$v = 122S - 1.713S^2$$

$$\frac{dv}{dS} = 0 = 122 - 3.426S$$

$$S = \frac{122}{3.426} = 35.6\ mi/h$$

$$v = 71.2D - 0.584D^2$$

$$\frac{dv}{dD} = 0 = 71.2 - 1.158D$$

$$D = \frac{71.2}{1.168} = 61.0\ \ pc/mi/\ln$$

The capacity is the product of the speed and density at capacity, or:

$$c = 35.6 * 61.0 = 2{,}172\ \ pc/h/\ln$$

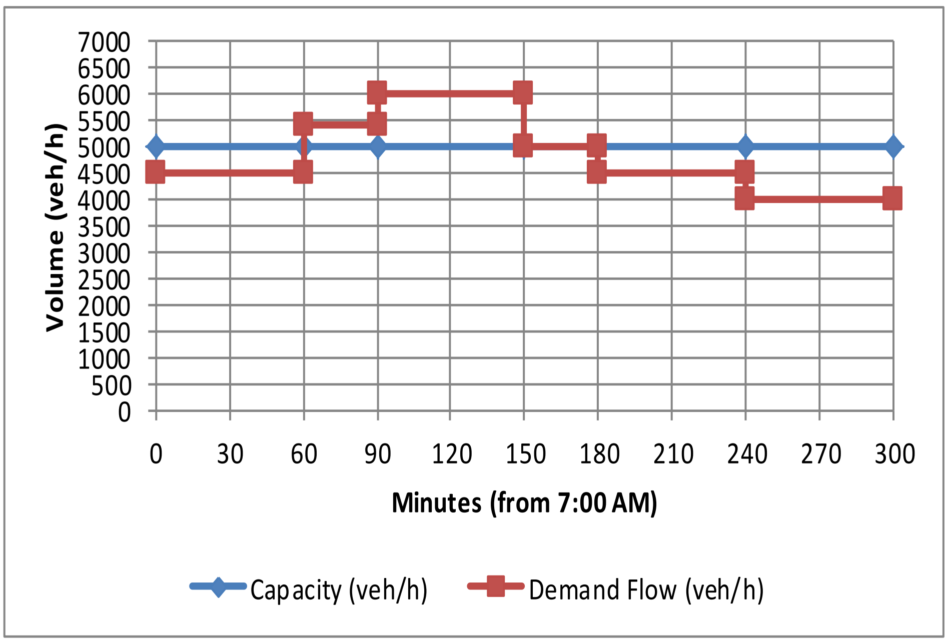
Solutions to Problems in Chapter 6

Concepts of Demand, Volume, and Capacity

Problem 6-1

To construct the graph, a spreadsheet is used. It is convenient to use a scale of time in minutes. Therefore, key times are converted to this scale as follows: 7:00 = 0 min, 8:00 = 60 min, 8:30 = 90 min, 9:30 = 150 min, 10:00 = 180 min, and 11:00 = 240 min. Time will be plotted on the X-axis. The Y-axis is the demand or capacity flow rate in veh/h. Capacity is a constant 5,000 veh/h throughout the study period. Demand varies with time, as stated. The plot follows.

Plot of Demand and Capacity vs. Time



a) At any given time, the size of the queue is measured as the area between demand and capacity lines. Queues are forming when the demand line is above the capacity line. When the demand line is below the capacity line, either there is no queue, or a previously formed queue is being dissipated. Note that the time scale is in minutes, and the flow scale is in veh/h. Queue computations must convert time in minutes to hours, or (conversely), demand and capacity to veh/min.

   Therefore, the size of the queue at various times is found as:

$$Q_t = Q_p + \left(\frac{t - t_p}{60}\right) * (v - c)$$

where: Q = size of queue at time t, vehs,

Q_p = size of queue at previous time check, vehs,

t	=	time, h,
t_p	=	time at previous check, h,
v	=	demand flow rate, veh/h, and
c	=	capacity flow rate, veh/h.

8:00 AM (t = 60 min)	Q = 0 vehs
8:30 AM (t = 90 min)	Q = 0 + 0.5 h*(5400-5000) = 200 vehs
9:30 AM (t = 150 min)	Q = 200+1.0 h*(6000-5000) = 1,200 vehs
10:00 AM (t = 180 min)	Q = 1,200 + 0.5h*(5000-5000) = 1,200 vehs
11:00 AM (t = 240 min)	Q = 1,200 + 1.0h*(4500-5000) = 700 vehs

b) At 11:00 AM, the queue stands at 700 vehicles. After 11:00 AM, the demand flow rate is 4,000 veh/h, while the capacity is 5,000 veh/h. Therefore, the queue will dissipate at a rate of 5,000-4,000 = 1,000 veh/h. Thus, the queue clears at 700/1,000 = 0.7 h *after* 11:00 AM. The queue, therefore, clears 0.7*60 = 42 minutes after 11:00 AM, or 11:42 AM.

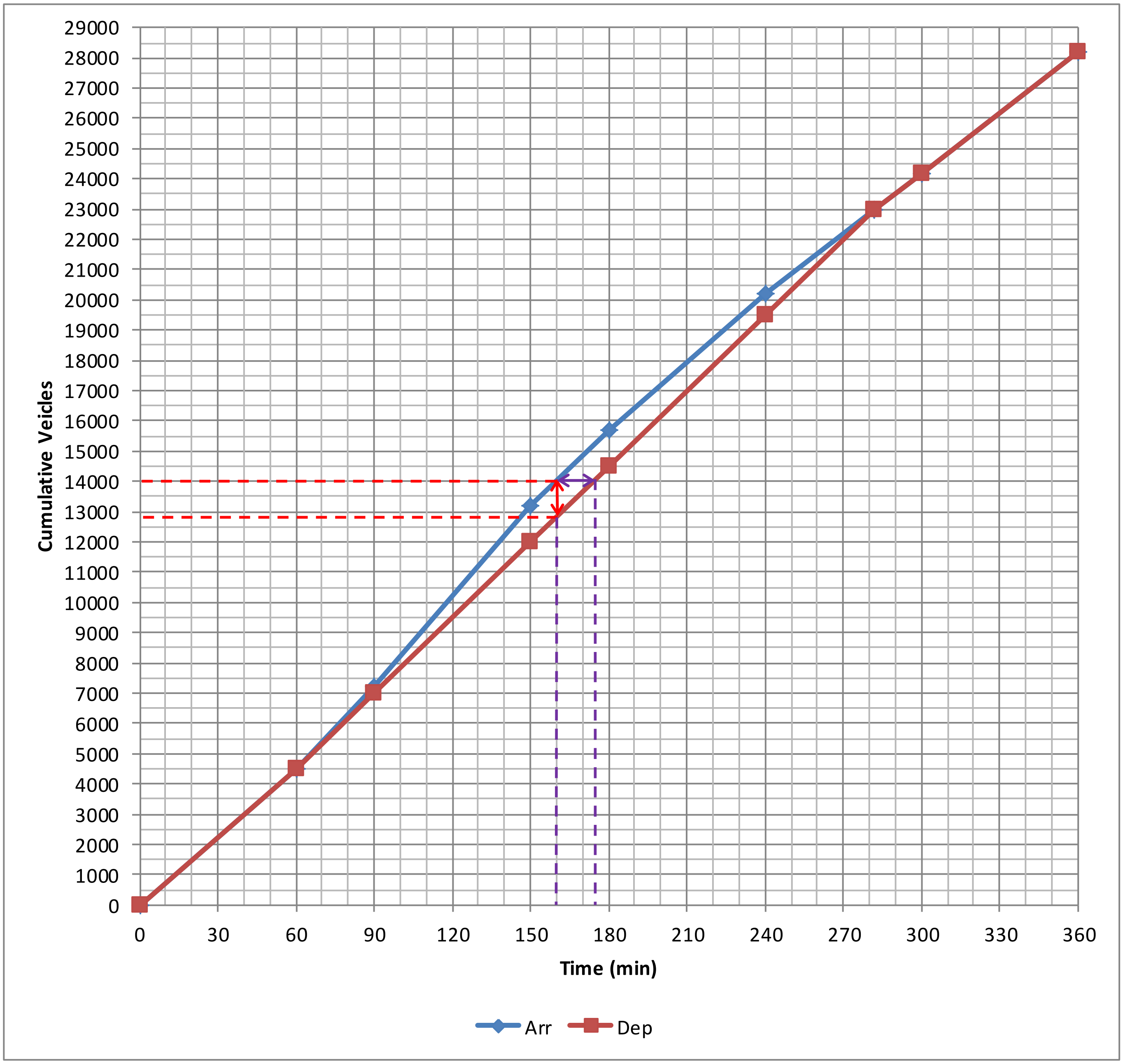
Problem 6-2

To construct a plot of cumulative arrivals vs. cumulative departures, a table of such arrivals and departures vs. time must be created. The key to doing this is the realization that until the time that the demand flow rate exceeds the capacity flow rate, arrivals will equal departures. Thereafter, departures are limited to the capacity flow rate, while arrivals will be dictated by the demand flow rate. When the queue is fully dissipated (at 11:42 AM in this case), subsequent arrivals will equal departures.

The table of cumulative arrivals and departures is shown below, and will be used to create the plot.

Time	**t (min)**	**Cumulative Arrivals (vehs)**	**Cumulative Departures (vehs)**
7:00 AM	0	0	0
8:00 AM	60	4,500	4,500
8:30 AM	90	4,500+(5,400/2) = 7,200	4,500+(5,000/2) = 7,000
9:30 AM	150	7,200+6,000 = 13,200	7,000 + 5,000 = 12,000
10:00 AM	180	13,200+(5,000/2) = 15,700	12,000+(5,000/2) = 14,500
11:00 AM	240	15,700+4,500= 20,200	14,500+5,000 = 19,500
11:42 AM	282	20,200+(4,000*0.7) = 23,000	19,500+(5,000*0.7) = 23,000
12:00 Nn	300	23,000+(4,000*0.3) = 24,200	24,200
1:00 PM	360	24,200+4,000 = 28,200	28,200

The plot of these follows.

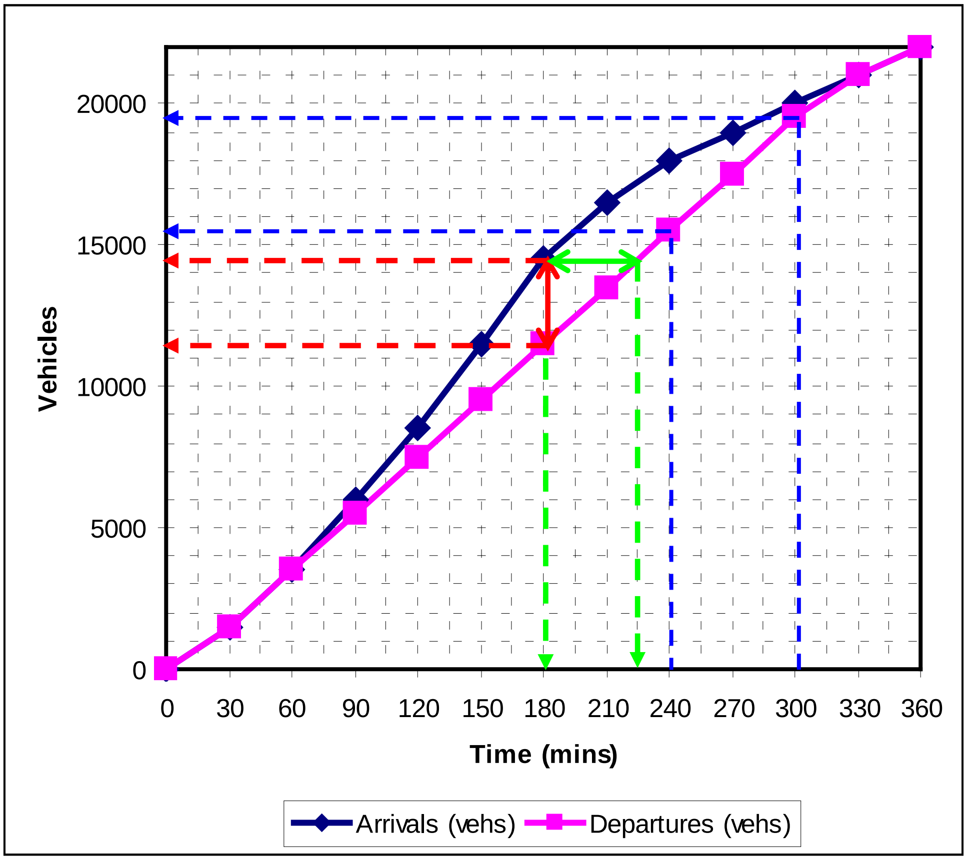


a) The queue size at any given time is the cumulative arrivals minus the cumulative departures. The queue reaches its maximum between 150 and 180 minutes. The size of the queue during this period is 14,000 – 12,800 = 1,200 vehicles (See graph, red lines)

b) The waiting time for any given vehicle is measured against the horizontal axis as the time of departure minus the time of arrival. Again, the maximum wait occurs between 150 and 180 minutes. The maximum wait time is about 175-150 = 125 min. (See graph, purple lines).

Problem 6-3

The solutions are illustrated on the graph below:



a. The capacity of the facility is represented by the departure rate during the period when demand exceeds capacity, i.e. anytime between 60 and 330 minutes. Using the hour between 240 and 300 minutes (blue lines), 19,500 – 15,500 = 4,000 vehicles are discharged (in one hour), or a rate of 4,000 veh/h.

b. Maximum queue size occurs anytime between 180 and 210 minutes. Using the 180 minute point (red lines), the maximum queue size is measured as the difference in vehicles arrived and departed at this time: 14,500 – 11,500 = 3,000 vehicles.

c. The maximum waiting time occurs to any vehicle arriving when the queue is at its maximum size, i.e., anytime between 180 and 210 minutes. It is measured as the difference between the arrival time and the departure time of any vehicle arriving within this period. Illustrated by the green lines, this is: 225 – 180 minutes = 45 minutes.

Problem 6-4

The two-lane blockage described exists for one hour, between 9AM and 10AM. During this time, there are only 2 lanes available for movement. After 10AM, the blockage is removed and there are 4 lanes available for movement.

A queue is established virtually immediately when the two lanes shut down (9AM). Thereafter, departures are determined by the queue discharge rate as opposed to stable capacity. Stable capacity returns only after the queue has been fully dissipated.

Thus, "capacity" during the first hour (with two lanes open) is 2*1800 = 3,600 veh/h. From 10 AM through the time the queue is cleared, four lanes are open, and the capacity is 4*1800 = 7,200 veh/h. The full capacity of 8,400 veh/h resumes only after the queue has been cleared.

a. The table below illustrates queue buildup and clearance:

Table: Queuing for Problem 4

Hour	Arrivals (veh/h)	Departures (veh/h)	Queue Size at End (veh)
9 – 10 AM	8,400	3,600	8,400-3,600 = 4,800
10 – 11 AM	8,000	7,200	4,800+8,000-7,200 = 5,600
11 AM – 12 Nn	7,000	7,200	5,600+7,000-7,200 = 5,400
After Noon	6,000	7,200	Queue diminishes at a rate of 1,200 veh/h

The maximum queue size is 5,600 vehicles, which occurs at 11 AM.

b. At 12 Noon, a queue of 5,400 vehicles continues to exist. From this point on, it will dissipate at a rate of 7,200-6,000 = 1,200 veh/h. It will take 5,400/1,200 = 4.5 h to fully dissipate. The queue will disappear at 4:30 PM in the afternoon.

Solutions to Problems in Chapter 7

The *Highway Capacity Manual*: History and Basic Concepts

Problem 7-1
Capacity is defined as the maximum sustainable flow rate that can be accommodated by a lane or roadway under prevailing roadway, traffic, and control conditions. The definition of a service flow rate is the same, except that the "maximum sustainable flow rate" applies to what can be accommodated while operating at a designated level of service. There is one value of capacity for a lane or roadway, and five service flow rates for levels of service A – E. LOS F is unstable, and no service flow rate is defined for it. For uninterrupted flow facilities or segments, capacity is the equivalent of the service flow rate for LOS E.

Problem 7-2
Capacity under "ideal" conditions refers to a capacity that could occur if all prevailing conditions were equivalent to the base conditions for each segment type. Such "ideal" conditions for uninterrupted flow segments include 12-ft lanes, 6-ft lateral clearances, and all passenger cars in the traffic stream. For interrupted flow segments, there are many "ideal" conditions to consider. Capacity under "prevailing" conditions refers to the impact of all of the actual conditions that exist on the segment, which may or may not be ideal.

Problem 7-3

a) Capacity for a freeway is equivalent to the service flow rate for LOS E, or 6,300 veh/h in this case.

b) Service volumes are computed from the service flow rates and the peak hour factor, as follows:

$$SV_A = SF_A*PHF = 3{,}000*0.90 = 2{,}700 \text{ veh/h}$$
$$SV_B = SF_B*PHF = 4{,}000*0.90 = 3{,}600 \text{ veh/h}$$
$$SV_C = SF_C*PHF = 4{,}800*0.90 = 4{,}320 \text{ veh/h}$$
$$SV_D = SF_D*PHF = 5{,}600*0.90 = 5{,}040 \text{ veh/h}$$
$$SV_E = SF_E*PHF = 6{,}300*0.90 = 5{,}670 \text{ veh/h}$$

Problem 7-4
Freeways have multiple lanes for the exclusive use of traffic in each direction and full control of access using on-ramps and off-ramps.

Multilane highways have multiple lanes for the exclusive use of traffic in each direction, but do not have full control of access. There are unsignalized intersections at grade and unsignalized driveway entrances.

Two-lane rural highways have one lane for use in each direction and no control of access. Their primary unique feature is that passing maneuvers are accomplished through the use of the opposing lane, requiring long sight distances and clear controls on where passing is permitted.

<u>Problem 7-5</u>
The capacity of the approach, per lane, is computed as:

$$c = s * \left(g/_C \right) = 1{,}200 * \left(40/_{70} \right) = 686\ pc/h$$

Because there are three lanes on the approach, the capacity of the approach in total is:

c = 686*3 = 2,058 pc/h

SOLUTIONS TO PROBLEMS IN CHAPTER 8

INTELLIGENT TRANSPORTATION SYSTEMS AND THEIR USE

Problem 8-1

The authors have taken the view that there are many resources now available on the web, and they should be used to supplement the textbook for at least two reasons: (1) more depth and detail than can be abstracted into a textbook, (2) knowledge that becomes available on the web, by additional links or by updates, between publication dates of the textbook's editions.

An ideal example is the 56-page report “History of Intelligent Transportation Systems”. Yes, the instructor has to be aware that assigning Problem 8-1 means the student must read a 56-page document, so a little guidance should go with the assignment:

- Read it completely, for in-class review and discussion. The document can be downloaded in PDF and other formats;
- Scan the table of contents and the document, focusing on the sidebars that emphasize key points and also looking for the history of themes that are newsworthy at the time of the assignment.

As a bit of history, the “ITS” effort began as the “IVHS” effort ----- Intelligent Vehicle Highway Systems. Indeed, ITS America was once IVHS America. One of the authors introduced a special topics graduate course based upon “IVHS” that evolved to “ITS”. As the years passed and a formal ITS Architecture was developed, additional courses were established.

The reference being read says on its Page 15 (page 25 of the PDF), “In 1994, the USDOT officially sanctioned the term ‘ITS’ as a replacement for IVHS, recognizing the multimodal nature of the activity and de-emphasizing the focus on technologies for vehicle guidance”.

The student should be aware that many ITS themes pre-existed what has become the unifying umbrella called ITS. Ramp metering, freeway surveillance and control, and computer control of traffic signals date back to the 1960's. But the ITS framework has certainly helped focus continuing attention, and advancement.

Problem 8-2

Reference [3] is https://www.pcb.its.dot.gov/ePrimer.aspx

Module 8 is “Electronic Toll Collection and Pricing” and “considers both the application and integration challenges and technologies available”.

Two thoughts for the course instructor:

1) Each of these problems can place a very heavy reading challenge on the student, in terms of pages and time required. Assignments should be made with this in mind;
2) The set of references in Chapter 8 can provide the basis for an entire course devoted to ITS and ITS-related topics. The instructor might consider this, for special topics or readings courses, or for a permanent addition to the curriculum.

The most relevant portion of Module 8 begins at its Figure 10, “Pricing Strategies”.

Before any class discussion, it is good to remember that in a broad conversation, some topics will cascade onto each other:

- What is the real, relative subsidy of different modes, particularly public transportation and roads used by private vehicles, goods movers, and for-hire vehicles?
- When does certain transportation become an essential feature of the community, such that its cost is part of the baseline existence of the community and not just an “add-on” or optional feature? In a dense urban area, isn’t public transportation essential, just as water and electricity and sewers? If so, why not include the basic infrastructure costs as part of the general obligation? But how about the <u>use</u> portion: individuals still pay for electricity consumed, water used, and such.
- So are roads an essential feature? If they are, then dense development can require so much roadway that it strangles the original concept of the area. Still, some level of access for some purposes --- goods, waste, and so forth --- is essential. In a competitive economy, so is access to business. But in what balance?
- If there is congestion pricing --- by any variation of its title --- does it fall equitably on the population, or are there groups that are trapped into autos and into fixed hours, and disproportionately affected?

All of these issues are valid from an overall planning view. It may be necessary to acknowledge these, but still try to focus the discussion on “We have today’s infrastructure, little additional space for construction, and long lead times. Faced with that, can pricing strategies (note that we just avoided the “congestion pricing” label) aid in fostering the economic well-being of the area and its people?”

Complicating the discussion is a related topic, cited in the problem statement ---- a significant source of revenue is the gasoline tax, as a fixed amount per gallon of fuel. But vehicle miles per gallon now varies widely, and the historic tax practice does not take into account vehicle occupancy – nor time of day of fuel usage. And with the growing presence of hybrid and electric vehicles, there are segments of the vehicle population that may not experience this tax at all.

Discussion point: perhaps current technology (electronic toll collection readers, connected vehicles, etc.) allow road use by time of day, local congestion levels, even occupancy to replace fuel taxation.

Problem 8-3

It will quickly be found that:

V2V = vehicle to vehicle communication
V2I = vehicle to infrastructure communication
I2V = infrastructure to vehicle communication

But as time goes on, it is likely that the literature will blend these into discussions of truly "autonomous vehicles" and make distinctions between those and "driver assistance systems" that require driver presence and awareness and even action. Market forces, public and legislative confidence, and safety results will influence these topics at a pace that writing a "right" answer to this question in a static "Solutions Manual" is not meaningful. But the rapid evolution of connected vehicles, truly autonomous vehicles, and related variants will certainly dominate the careers of the current students.

Problem 8-4

Reference [6] is https://www.fhwa.dot.gov/cadiv/segh

The first time we entered it (without the "s"), we ended up at the wrong site and had to search for "Systems Engineering Guidebook for ITS" and got to the right place.

Once the V Diagram is reached, clicking on each step provides much information, including "Objective" and "Description" of the step.

Students will have to be reminded that plagiarism is to be avoided ----- one of the ways is to clearly put quotes around text that is transposed into their homework, and to cite the source.

The purpose of assigning this problem is to force the students to remember the V Diagram, which will probably come up repeatedly in their professional practice.

Problem 8-5

The starting points are

- The web search on "probe vehicles", and
- A keyword search of Reference [1] for "probe" and "probe vehicle"

The concept that will be revealed to the student is the idea that a finite number of vehicles (say 500 or 1,000) are equipped with transponders and their travels recorded as they traverse the network --- a history of time and location stamps, often transmitted wirelessly to a central location. From that information, congestion can be profiled, path selections (and path selection differences, from day to day), travel time can be estimated, and so forth.

A variation on this theme is to use on-board devices such as ETC units or phone or (as connected vehicle technology becomes more pervasive) vehicle ID information to garner the same sort of information from "scrubbed" data to preserve privacy, with a larger number of de facto probe vehicles.

Truck and bus fleets increasingly use GPS location in real time, and thus become a source of probe data information. The same is true of taxi fleets in some locations. If anything, the trend is to more and more "probe" vehicles being sampled, to the point that the term may get overused or fall into disuse.

If every vehicle is made into a de facto probe vehicle, what is the meaning of the term?

As path selection information become more available, it may be possible to construct O/D paths within a network, take note of the path changes in response to traffic conditions (assuming that information is available concurrently, or deduced from travel times), and build O/D information and related path selection into simulation models.

Problem 8-6

There are two distinct paths to be discussed:

1) The traditional means of distributing mailback questionnaires or conducting interviews on-site (toll booths, inspection stations, truck stops);
2) Advanced methods that depend upon automated collection of license plates, ETC information, and/or USDOT numbers on the sides of trucks.

The first path is plagued by small sample sizes, due to number of on-site interviews or due to mail-back response rates. As toll plazas move to cashless toll collection and higher speed passage, the very opportunity to hand out questionnaires is ceasing to exist.

At the same time, reaching drivers or owners through using their ETC information is often viewed negatively, because the issuing authority does not want to encounter privacy and trust issues with those who register for ETC.

The ETC numbers are certainly attractive --- in principle --- for O/D, if there is a fairly dense network of ETC readers and a limited number of major paths to the destination zones (for instance, counties), as well as origin zones. But at its best, this still does not address the specific cargo being carried, or whether the truck was empty (deadheading).

One authority has used the USDOT number on the truck, identified the owner through the SAFER data base (more, in the next paragraph), and done phone interviews based upon that information. Response rates were higher than earlier means.

The Federal Motor Carrier Safety Administration (FMCSA) is one of the administrations of the USDOT. As part of its work, it maintains the Safety and Fitness Electronic Records (SAFER) System, often referred to as the "SAFER database". For information, see

https://safer.fmcsa.dot.gov/about.aspx

https://safer.fmcsa.dot.gov/saferhelp.aspx

Truckers can obtain a USDOT number for their vehicles through the SAFER data base. These numbers are displayed on the side of the truck. The database contains information on the trucker, the number of power units (single body trucks, tractors, etc.) owned, and other information *including cargos carried.* But the USDOT number is assigned to multiple vehicles owned by that registered trucker, so it is not unique to the vehicle. And "cargos carried" can be a range, and not reflect the cargo carried on a specific day.

But the SAFER database is good for certain descriptive statistics (fleet sizes, range of cargos, etc.) and as a source of owner information, which can be used in a follow-up survey. The authors are of the opinion that surveys that once focused on a facility (or set of facilities, such as river crossings in an area) can be re-cast as a survey of owners that use those facilities, and perhaps be structured so that it focuses on the characteristic uses of the facility or facilities by those owners --- types of cargo, O/D, time of day, empty or not, and so forth.

Does ITS technology provide the path to improved surveys of this sort? Yes, and the potential may be best realized by fusion of different data bases (ETC, USDOT numbers, etc.) and some outreach for things not yet known by the technology (full, partial load, empty).

Problem 8-7

Section 8.6 addresses "Connected Vehicle Pilot Studies" as of late 2017. The intent of this question is simply to have the student update the information to the year of the assignment, from the web and perhaps from published USDOT reports, TRB papers, and other sources.

Problem 8-8

At the time of the 5th edition, inductive loops cut into the pavement were still a very common way of detecting traffic presence, passage, and even estimating speed. Smaller magnetic units were becoming more common, as was wireless transmission of traffic data from such detectors.

But the concept of using strategically mounted cameras to view a larger area (an intersection approach, a freeway section, or several intersection approaches), combined with software that allowed the user to place multiple "virtual" detectors on top of the screen image was becoming a powerful alternative. The defined virtual detectors could mimic loops near the stopline or upstream, include some "area" detector by lane or other, and achieve rather intense detectorization with one instrument and with no pavement cuts. Regular or infrared cameras could be used, and algorithms to deal with sun glare were being reported and/or used.

The intent of this question is simply to have the student update the information to the year of the assignment, because the technology is evolving. Class discussions on cost-effectiveness, user-friendliness of the drag-and-drop detector locations, and "Why still use loops?" can be takeoffs from their results.

Solutions to Problems in Chapter 9

Traffic Data Collection and Reduction Methodologies

Problem 9-1

The counting period is 15 minutes, but actual counts are for 12 out of 15 minutes. Thus, all counts must be expanded by 15/12 = 1.25, rounded, of course, to the nearest whole vehicle. Since counts in each lane are for alternate period, the missing counts must be interpolated from the counts in adjacent periods, except for blank first and last periods, which must be extrapolations, not interpolations. The table below illustrates these computations.

Table: Expanded and Interpolated/Extrapolated Counts

Time of Count	Eastbound			Westbound			Total
	Lane 1	Lane 2	Total	Lane 1	Lane 2	Total	
4:00-4:15	**423**	450	873	**346**	388	734	1607
4:15-4:30	438	*463*	900	356	*401*	757	1657
4:30-4:45	*451*	475	926	*366*	413	778	1704
4:45-5:00	463	*469*	932	375	*419*	794	1726
5:00-5:15	*447*	463	910	*363*	425	788	1697
5:15-5:30	431	*444*	875	350	*407*	757	1632
5:30-5:45	*416*	425	841	*338*	388	725	1566
5:45-6:00	400	**406**	806	325	**368**	693	1499

Notes: In lane counts: italics = interpolated counts; bold=extrapolated counts.

Note that a spreadsheet was used for computations. Thus, "round-offs" may result in what appears to be an error of "1" in a sum for both lanes in a direction, or for the total volume in both directions.

Problem 9-2

X	Y	Z = (X)(Y)
# Axles per Vehicle	**Number of Axles Per Vehicle**	**Number of Axles Observed**
2	157	314
3	55	165
4	50	200
5	33	165
6	8	48
Total # Axles Observed =		**892**
# vehicles =	303	
average # axles per vehicle =		**2.94**

11,250	actuations on road tube, 24 hours
2.94	average # axles per vehicle
3,821	**estimated vehicles, 24 hours**

Problem 9-3

From Figure 9.7, the trigonometric relationships are defined by Equations 9-3. The effective distance over which travel times are actually observed is computed as:

$$d_{eff} = d_1 Tan(70) = 50 * 2.7475 = 137.38 \ ft$$

The speed of a vehicle with a travel time of 2.15 s is found as:

$$S = \frac{137.38}{2.15} = 63.9 \ ft/s = \frac{63.9}{1.47} = 43.5 mi/h$$

Knowing the exact parallax error is difficult at best, and requires some additional field measurements. When an observation position is established and the exit boundary is marked, test observations are made to establish what viewing angle must be used to observe exiting vehicles. Thus, that actual marked trap (d) is measured and marked, then a viewing angle is established, from which the distance d_{eff} can be computed. The exact parallax error is then $d - d_{eff}$. That is why it is difficult to measure speeds manually in this manner. Radar or other meters are far easier to use.

Problem 9-4

This problem has its own merits, but is also a good “teaching moment” for the instructor on the subject of transitioning from an older method to a newer method.

On the surface, this issue can be viewed simply as “We always did it this way, and it was always good enough” followed by “How do we know the new method is equally good?”

While resistance to change is normal, there is also a substantial issue imbedded in accepted practices, and the reports that are generated from them, and submitted to various authorities and to the public. For instance, the annual Fall ATR Cordon Count may have become institutionalized, and used by the commissioner’s office in setting priorities, by the mayor’s office in allocating or authorizing budgets, and by the media. A shift to a new method can look at abandoning decades of history.

> We are going to digress for a moment, to provide the instructor with some history and perhaps perspective: the TRB Committee responsible for the *Highway Capacity Manual* once discussed to changing levels of service from “A” to “F” to a continuum of numbers, say “0” to “10” with “10” as level of service A, and facilities assigned numbers to one decimal place, such as 7.4.
>
> The discussion ended with the resistance of experienced persons who pointed out that (1) it had taken decades to educate elected officials and others to the “A” through “F” system, and (2) people were relating to it, based upon grades from their school days. It was not simply inertia, but a hard-won acceptance of a system that communicated effectively.
>
> People do talk about “the middle of LOS C” and such these days, but there has been no movement to “+” and “-“ grades. If such a movement began, it would have to be remembered that the present levels of service are defined by boundaries, so a logical question is whether C+, C, and C- should all fit within the existing LOS C. And that would have very practical implications, because the commissioner may have to explain to a legislative committee how --- despite funding provided in recent years --- the quality of the system suddenly slipped to a lower grade

> One more digression:
>
> - Early drafts of the 1965 HCM did not use the letter grade convention, but rather assigned numbers. And there was some discussion on which number represented the best condition and which the worst.

> - The answer to "How many levels of service are there?" depends upon the year. The 1965 HCM defined <u>five</u> levels of service "A" to "E"; "F" was not a level of service but rather a failure to provide a level of service. In the 1985 HCM and beyond, there are six levels of service, including the all-too-common level of service F.

Back to the subject at hand: moving away from an accepted method often has intrinsic problems, beyond simple technical accuracy. But it also has sound and realistic questions of comparable technical accuracy.

One of the authors was involved in such a deliberation, that sometimes took on the nature of a debate. Some participants started with the position that "The new method has to give the same answers as the old" and that slowly evolved to "The new method has to have at least equal precision as the old, and be indistinguishable from it".

A "bake-off" comparison was then designed, with several methods to be used concurrently at the same location(s) over a range of conditions (that is, traffic volumes). The methods were:

- Method 1: ATR's, with road tubes laid down and checked as traditional;
- Method 2: permanent installation of 1 or 2 side-fire microwave detectors, with the number dependent upon the local geometry;
- Method 3A: manual counts, by an experienced observer;
- Methods 3B: manual counts, by a second experienced observer.

It would have also been good to take a video recording of the traffic, but this was not done. "Method 2" should be considered as whatever the proposed alternative might be, perhaps automated counting via cameras.

It was fortuitous that there were two people at the scene, assigned to counting (rather than have one on standby). As it turned out, the results showed that the three methods gave comparable results <u>and</u> that the two manual counts were not identical. Indeed, they differed from each other as much as they each differed from the ATR's and Method 2.

The expectation for the student would be that he/she touch on some of the above and be introduced to more in any class discussion of the problem. It would also be valid for the student to raise as relevant points

- The range of traffic and geometries to be considered as test sites, given the conditions stated at the end of the problem description;
- Raise the question of how much data is enough, with the better answers addressing the need to test the hypothesis that there is no difference between

two regression lines or that one method can be used to make adjustments to the other (if necessary; the better result would be to not reject the hypothesis of “no difference” at a level of significance of 0.05 or even better. But this expectation has to be tempered by how much statistics students should have learned in prior courses.

Problem 9-5

This problem is straight-forward, and the information cited is now common on agency websites. If that is not the case locally at the course location, the instructor may wish to refer the students to illustrative sites for the State DOT.

Problem 9-6

The web site for Miovision is https://miovision.com/

The authors chose this company because it is the highest visibility vendor of this product class at the time the 5th edition of the text was prepared. Further, the web site has some very fine educational materials for the student, describing a range of devices and solutions. But significant competitors tend to emerge, and the instructor may suggest alternatives to also look at. Attention should not be limited to the United States.

Given the wording of the question, the student is expected to be providing an update in some future year. We cannot be more specific than the guidance below in this solution manual, which is being written concurrent with the 5th edition of the textbook.

To begin a Google search, the authors used “traffic cameras for traffic counts” and came up with a good number of sites, in January of 2018.

Regarding literature, the authors began with “transportation literature search” but also knew that the Transportation Research Board (TRB) site at https://trid.trb.org/ would be of use, USDOT at https://rosap.ntl.bts.gov/ and https://ntl.bts.gov/ would be of use, and that the articles and advertisements in such places as the *ITE Journal* and *ITS/International* would be good sources.

TRB *Transportation Research Circular E-C194* (March 2015) is titled “Literature Searches and Literature Reviews for Transportation Research Projects” and subtitled “How to Search, Where to Search, and How to Put It All Together: Current Practices”. It can be downloaded by the student.

The student should be expected to make distinctions between automated traffic counting for reports and analysis (such as signal retiming, traffic impact work, and such) and for real-time control of traffic.

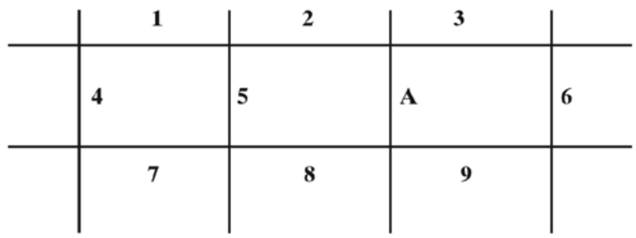
Another aspect for the student to comment upon might be whether the camera can collect in all weather conditions, how accurate it has been found to be, and how cost-effective the solution is deemed to be.

As of this writing, some systems are sold, with the vendor also offering data processing services for a fee. Rental is another option, also with data processing services available for a fee. Third-party vendors that provide just the data processing service (camera files to Excel, for instance) are emerging.

Solutions to Problems in Chapter 10

Volume Studies and Characteristics

Problem 10-1



The problem calls for estimating a total 12-hour volume for the study data shown. There is one control-count station (Station A) and 9 coverage-count stations (Stations 1-9). There are several issues that must be addressed in the estimation process:

- Data was taken in three four-hour periods: 8 AM to 12 Noon, 12 Noon to 4 PM, and 4 PM to 8PM. To allow for movement of data crews, however, actual counts were taken for 3.75 hours out of each 4-hour period. All counts, therefore, must be multiplied by 4.00/3.75 = 1.067 to estimate the actual 4-hr counts.

- Counts were taken using road tubes, and thus represent axle-counts, not vehicle-counts. Sample data on traffic composition must be used to estimate the average number of axles per vehicle, which can than be used to convert axle-counts to vehicle-counts.

- Counts taken during one 4-hour period must be expanded to estimate counts for the 12-hour target period.

- Counts were taken across three days. All counts must, therefore, be adjusted to reflect the average day of the count.

These conversions can be done in almost any order, and are best done using a spreadsheet. As all results must be rounded to the nearest vehicle, the order of computations and the rounding mechanism used may cause small discrepancies in final answers. In this solution, rounding is done *only* in the final step, although most of the spreadsheet tables will appear to be rounded at each step.

Table 1, which follows, computes the average number of axles per vehicle from the sample data given in the problem statement. The total number of axles observed is divided by the total number of vehicles observed to determine the conversion factor.

Table 1: Computing the Average Number of Axles Per Vehicle

Vehicle Class	Vehicles Observed	Axles Observed
2-axle	1,100	2,200
3-axle	130	390
4-axle	40	160
5-axle	6	30
Total	**1,276**	**2,780**

Average Axles/Vehicle = 2,780/1,276 = 2.18

The data from the Control Count Station A must now be manipulated to produce conversion values for coverage counts. Two conversions must be conducted: a) from 4-hr counts to 12-hr counts, and b) from 12-hr counts on a particular day to 12-hr counts representing the average of the three days of the study.

The first is accomplished by calibrating the percentage of 12-hour volume that occurs in each 4-hour period. For each day of the study, the percentage is computed as $(V_4/V_{12})*100$. There will be different values for each day of the study. These can be applied separately to coverage counts on the same day, or the average percentages can be applied to all three days.

The second conversion is accomplished by calibrating "daily variation factors" for each of the three days of the study. These factors are defined as V_{AVE}/V_{DAY}. The calibration of these values can be based directly on the 3.75-hr axle-counts given in the problem statement. These values *could* be converted to 4-hr vehicle-counts and used, but the conversions would affect every number equally, and none of the conversion values would be changed. Table 2 illustrates the computation of these conversion values in spreadsheet form.

In terms of expanding counts from 4 hours to 12 hours, the percentages do not vary greatly for each day of the study. Therefore, percentages based upon the average data will be used.

Coverage counts are now expanded to full 12-hour vehicle counts in Table 3, using the following equation:

$$V_{12i} = \frac{1.067 V_{3.75i} * DF_j}{p_k}$$

Where: V_{12i} = 12-hour vehicle count for Station i, vehs

$V_{3.75i}$ = 3.75-hour axle count for Station i, axles

1.067 = expansion factor, 3.75 hrs to 4 hrs

DF_j = daily adjustment factor for day j

p_k = percentage of volume occurring during time period k, expressed as a decimal

Table 2: Calibration of Conversion Values from Control-Count Data

Day	Time Period				Daily Adj Factor
	8:00-11:45	12:00-3:45	4:00-7:45	Total	
Axle Counts					
Monday	3,000	2,800	4,100	**9,900**	**1.118**
Tuesday	3,300	3,000	4,400	**10,700**	**1.034**
Wednesday	4,000	3,600	5,000	**12,600**	**0.878**
Total	**10,300**	**9,400**	**13,500**	***33,200***	**11,067**
Percent of 12-Hour Total					
Monday	30.30%	28.28%	41.41%	**100.00%**	NA
Tuesday	30.84%	28.04%	41.12%	**100.00%**	NA
Wednesday	31.75%	28.57%	39.68%	**100.00%**	NA
Total	**31.02%**	**28.31%**	**40.66%**	**100.00%**	NA

Table 3: Expansion and Adjustment of Coverage Counts to 12-Hour Vehicle-Counts

Station	Day	Time	Axle Count	Exp to 4 Hr	Exp to 12-Hr	Daily Adj	12-Hr Vehs
1	Monday	8:00-11:45	1,900	1.067	0.3102	1.118	**7,307**
2	Monday	12:00-3:45	2,600	1.067	0.2831	1.118	**10,956**
3	Monday	4:00-7:45	1,500	1.067	0.4066	1.118	**4,401**
4	Tuesday	8:00-11:45	3,000	1.067	0.3102	1.034	**10,670**
5	Tuesday	12:00-3:45	3,600	1.067	0.2831	1.034	**14,030**
6	Tuesday	4:00-7:45	4,800	1.067	0.4066	1.034	**13,024**
7	Wednesday	8:00-11:45	3,500	1.067	0.3102	0.878	**10,570**
8	Wednesday	12:00-3:45	3,200	1.067	0.2831	0.878	**10,589**
9	Wednesday	4:00-7:45	4,400	1.067	0.4066	0.978	**11,292**

Problem 10-2

Daily variation factors may be computed as:

$$DF = \frac{V_{AVE}}{V_{DAY}}$$

Where: V_{AVE} = average daily count for all days of the week, vehs

V_{DAY} = average daily count for each day of the week, vehs

These computations are carried out in Table 4.

Table 4: Calibration of Daily Adjustment Factors

Day	Ave Vol	Daily Factor
Sunday	3,500	**1.155**
Monday	4,400	**0.919**
Tuesday	4,200	**0.963**
Wednesday	4,300	**0.940**
Thursday	3,900	**1.037**
Friday	4,900	**0.825**
Saturday	3,100	**1.304**
TOTAL	28,300	
AVERAGE	**4,043**	

<u>Problem 10-3</u>

a) 5 minutes or 15 minutes. Count 4 of 5 or 13 of 15.
The counting period and the actual count time must be multiples of 1 minute.

b) 6 minutes or 18 minutes. Count 4.5 of 6 or 15 of 18. The counting period and the actual count time must be multiples of 90 seconds or 1.5 minutes.

c) 6 minutes or 18 minutes. Count 4 of 6 or 16 of 18. The counting period and the actual count time must be multiples of 2 minutes.

<u>Problem 10-4</u>

Daily adjustment factors are based upon the data in the problem statement. The factors, which use the same equation noted in Problem 10-2, are based upon the average of the 4 weeks of data provided.

Monthly adjustment factors are based upon the data in the problem statement, and are computed using the following equation:

$$MF_i = \frac{AADT}{ADT_i}$$

Where: MF_i = monthly adjustment factor month i

AADT = average annual daily traffic , vehs/day (estimated as the average of 12 monthly ADTs)

ADT_i = average daily traffic for month I, vehs/day

Daily adjustment factors are calibrated in Table 5. Monthly adjustment factors are calibrated in Table 6. Monthly variation factors must be themselves "adjusted" to reflect the *middle* of each month. This is done graphically in Figure 1.

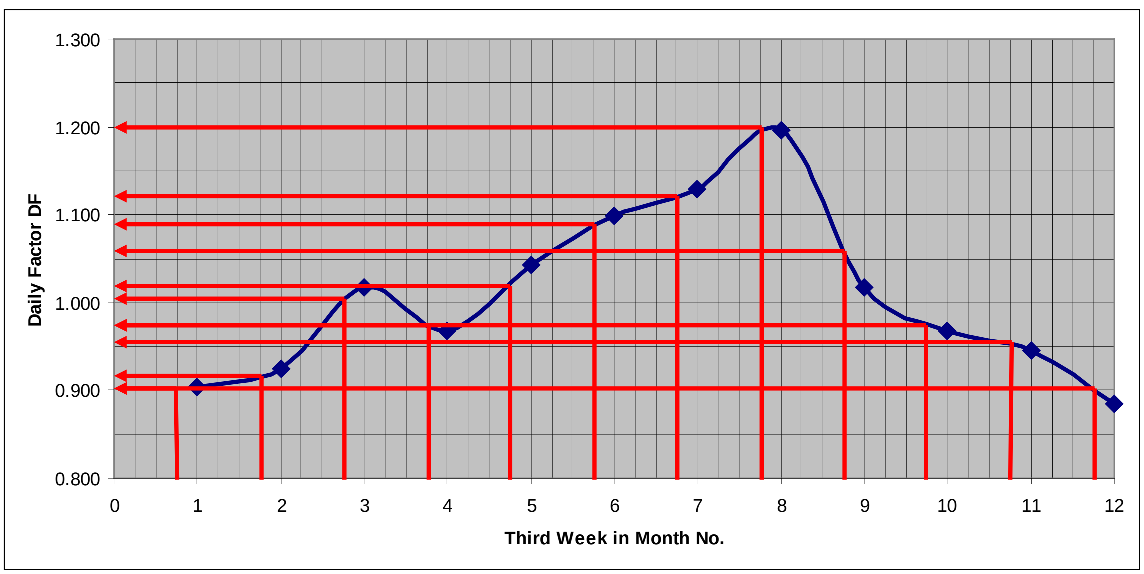
Table 5: Daily Adjustment Factors Calibrated

First Week In:	Day of the Week							
	Monday	**Tuesday**	**Wednesday**	**Thursday**	**Friday**	**Saturday**	**Sunday**	**TOTAL**
January	2,000	2,200	2,250	2,000	1,800	1,500	950	12,700
April	1,900	2,080	2,110	1,890	1,750	1,400	890	12,020
July	1,700	1,850	1,900	1,710	1,580	1,150	800	10,690
October	2,100	2,270	2,300	2,050	1,800	1,550	1,010	13,080
TOTAL	*7,700*	*8,400*	*8,560*	*7,650*	*6,930*	*5,600*	*3,650*	
AVERAGE	**1,925**	**2,100**	**2,140**	**1,913**	**1,733**	**1,400**	**913**	12,123
DF	0.900	0.825	0.809	0.906	1.000	1.237	1.898	**1,732**

Table 6: Monthly Adjustment Factors Calibrated

Third Week In	Ave 24-Hr Count	Monthly Factor
January	2,250	**0.904**
February	2,200	**0.924**
March	2,000	**1.017**
April	2,100	**0.968**
May	1,950	**1.043**
June	1,850	**1.099**
July	1,800	**1.130**
August	1,700	**1.196**
September	2,000	**1.017**
October	2,100	**0.968**
November	2,150	**0.946**
December	2,300	**0.884**
TOTAL	**24,400**	
AVERAGE	**2,033**	

Figure 1: Monthly Adjustment Factors "Adjusted"



In Figure 1, Monthly Factors are plotted at the end of the 3rd week of each month, when the counts were taken. Ideally, factors should represent the "middle" of the month, which is usually at the end of the 2nd week. The graph approximates four weeks per month (except for Feb, there are actually 4 + a fraction). The end of the 2nd week can, therefore, be approximated as one week earlier than the actual count. The factors for the "middle" of the month are read from the graph, and are entered in Table 7.

Table 7: Monthly Factors Adjusted for the Middle of the Month

Month	Adjusted Monthly Factor MF
January	0.900
February	0.920
March	1.005
April	0.975
May	1.020
June	1.090
July	1.120
August	1.200
September	1.060
October	0.975
November	0.955
December	0.900

Problem 10-5

The four control count stations shown in the problem statement are proposed to form a single "group" for the purpose of calibrating Daily Adjustment Factors DF. To be an appropriate grouping, the "average" factor for each day of the week cannot differ from the factors at each station by more than ± 0.10. The grouping is evaluated in Table 8.

Table 8: Average Daily Factors for Group and Assessment

Station	Monday	Tuesday	Wednesday	Thursday	Friday	Saturday	Sunday
1	1.04	1.00	0.96	1.08	1.17	0.90	0.80
2	1.12	1.07	0.97	1.06	1.02	0.87	0.82
3	0.97	0.99	0.89	1.01	0.86	1.01	1.06
4	1.01	1.00	1.01	1.09	1.10	0.85	0.86
Total	**4.14**	**4.06**	**3.83**	**4.24**	**4.15**	**3.63**	**3.54**
Average	**1.035**	**1.015**	**0.9575**	**1.06**	**1.0375**	**0.9075**	**0.885**
OK Range	0.935-1.135	0.915-1.115	0.8575-1.0575	0.96-1.16	0.9375-1.1375	0.8075-1.0075	0.785-0.985

Obviously, three of the factors lie outside the acceptable range. It appears that Station 3 should be eliminated. Assuming that they are still spatially contiguous, Stations 1, 2, and 4 may be grouped, and must again be tested, as shown in Table 9.

Table 9: Re-Grouped Stations Tested

Station	Monday	Tuesday	Wednesday	Thursday	Friday	Saturday	Sunday
1	1.04	1.00	0.96	1.08	1.17	0.90	0.80
2	1.12	1.07	0.97	1.06	1.02	0.87	0.82
4	1.01	1.00	1.01	1.09	1.10	0.85	0.86
Total	**3.17**	**3.07**	**2.94**	**3.23**	**3.29**	**2.62**	**2.48**
Average	**1.057**	**1.023**	**0.980**	**1.077**	**1.097**	**0.873**	**0.827**
OK Range	0.957-1.157	0.923-1.123	0.880-1.080	0.977-1.177	0.997-1.197	0.773-0.973	0.727-0.927

The re-grouping meets the acceptability criteria, and would be used.

Problem 10-6

Data from coverage counts within the control area depicted in text Table 10-11 are given. We are asked to estimate the annual VMT for each section counted. To do this, the AADT at each location must be estimated. The following equations are used:

$$AADT = V_{i,j} * DF_i * MF_j$$
$$Annual\ VMT = AADT * L$$

Where: AADT = average annual daily traffic, vehs/day

V_{ij} = count taken on day i in month j, vehs/day

DF_i = daily factor for day i

MF_j = daily factor for month j

L = length of the study segment, mi

These computations are illustrated in Table 10.

Table 10: Estimated AADT and VMT for Stations in Problem 8-6

Station	Length (mi)	Day	Month	DF (Table 9.11)	MF (Table 9.11)	Daily Vol (vehs/day)	AADT (vehs/day)	Veh Miles
1	3.0	Wed	March	1.108	1.100	9,120	**11,115**	**33,346**
2	2.7	Tue	September	1.121	0.884	10,255	**10,162**	**27,438**
3	2.5	Fri	August	1.015	0.882	16,060	**14,377**	**35,943**
4	4.6	Sun	May	0.789	0.949	21,858	**16,366**	**75,286**
5	1.8	Thu	December	1.098	1.114	9,508	**11,630**	**20,934**
6	1.6	Fri	January	1.015	1.215	11,344	**13,990**	**22,384**

Problem 10-7

The problem statement gives the data from an origin and destination study. Only sample measurements are made, and an estimate of the actual O-D counts for the period of the study is needed. The O-D matches can be adjusted so that the row totals (origins) are correct, or so that the column totals (destinations) are correct. The accepted methodology is to average these two approaches until *all* row and column totals are within ±10% of the observed volumes at each origin and destination. This is an iterative process.

Each row and each column has an adjustment factor that will resolve the row or column totals. The actual adjustment of the O-D matches (cells of the table) are computed as follows:

$$OD_{i+1} = OD_i * \left(\frac{F_{oi} + F_{di}}{2} \right)$$

Where: OD_{i+1} = OD volume for the i+1 iteration, vehs

OD_i = OD volume for the ith iteration, vehs

F_{oi} = adjustment factors based upon resolving origin totals, ith iteration

F_{di} = adjustment factors based upon resolving destination totals, ith iteration

In each iteration, the adjustment factors are re-computed as follows:

$$F_{oi} = \frac{V_o}{\sum_i OD_{ij}}$$

$$F_{di} = \frac{V_d}{\sum_j OD_{ij}}$$

Where: V_o = total observed volume at origin "o" (vehs)

V_d = total observed volume at destination "d" (vehs)

OD_{ij} = OD matches for origin "i" and destination "j", vehs

These computations are shown in Table 11. Iterations are continued until all origin and destination totals are resolved to ±10%.

Table 10: Origin and Destination Adjustments

Destination Station	Origin Station					Destination Sum	Destination Vol	F_d
	1	2	3	4	5			
1	50	120	125	210	75	**580**	1,200	**2.069**
2	105	80	143	305	100	**733**	2,040	**2.783**
3	125	100	128	328	98	**779**	1,500	**1.926**
4	82	70	100	125	101	**478**	985	**2.061**
5	201	215	180	208	210	**1,014**	2,690	**2.653**
Origin Sum	**563**	**585**	**676**	**1,176**	**584**	***3,584***		
Origin Vol	1,820	1,225	1,750	2,510	1,110		**8,415**	
F_o	**3.233**	**2.094**	**2.589**	**2.134**	**1.901**			
FIRST ITERATION								
Destination Station	**Origin Station**					**Destination Sum**	**Destination Vol**	**F_d**
	1	**2**	**3**	**4**	**5**			
1	133	250	291	441	149	**1,264**	1,200	**0.950**
2	316	195	384	750	234	**1,879**	2,040	**1.086**
3	322	201	289	666	187	**1,666**	1,500	**0.901**
4	217	145	232	262	200	**1,057**	985	**0.932**
5	591	510	472	498	478	**2,550**	2,690	**1.055**
Origin Sum	**1,579**	**1,302**	**1,668**	**2,617**	**1,249**	***8,415***		
Origin Vol	1,820	1,225	1,750	2,510	1,110		**8,415**	
F_o	**1.152**	**0.941**	**1.049**	**0.959**	**0.889**			
SECOND ITERATION								
Destination Station	**Origin Station**					**Destination Sum**	**Destination Vol**	**F_d**
	1	**2**	**3**	**4**	**5**			
1	139	236	291	421	137	**1,224**	1,200	0.980
2	353	198	410	767	231	**1,959**	2,040	1.041
3	331	185	282	619	168	**1,584**	1,500	0.947
4	226	136	230	248	182	**1,023**	985	0.963
5	653	509	496	501	465	**2,625**	2,690	1.025
Origin Sum	**1,703**	**1,264**	**1,709**	**2,556**	**1,183**	***8,415***		
Origin Vol	1,820	1,225	1,750	2,510	1,110		**8,415**	
F_o	1.069	0.969	1.024	0.982	0.939			

Note that in each iteration, any origin or destination adjustment factor that is less than 0.900 or more than 1.100 indicates that there is still a discrepancy greater than 10% in origin or destination totals. Iterations are continued until the adjustment factors for both origins and destinations all lie within a range of 0.900 to 1.100.

Solutions to Problems in Chapter 11

Speed, Travel Time, and Delay Studies

Problem 11-1

Parts a and b:

To plot a frequency distribution curve and a cumulative frequency distribution curve, a frequency distribution table must be constructed from the data given. Table 1 provides this. Then:

- The Frequency Distribution Curve (FDC) is plotted as the % Vehicles in Group vs. the Middle Speed of the group.
- The Cumulative Frequency Distribution Curve (CFDC) is plotted as the Cum % Vehicles in Group vs. the Upper Speed of the speed group.
- The *median speed* is the 50^{th} percentile speed from the CFDC.
- The *modal speed* is estimated as the peak of the FDC.
- The *pace* is the 10-mi/h increment in speed that captures the highest percentage of observed speeds compared to any other 10-mi/h increment.
- The *percent vehicles in the pace* are found by finding the percentile speed representing the boundaries of the pace, and subtracting their values.

Figure 1 shows the two curves, and the solution for the values called for in Part b of the question.

From Figure 1:	Mode	=	42.0 mi/h
	Median	=	42.0 mi/h
	Pace	=	37.5 – 47.5 mi/h
	% Veh in Pace =		72% - 24% = 48%

Table 1: Frequency Distribution Table for Speed Data

Speed Group (mi/h)		Middle	Observed	Percent	Cum %		
Low Speed	High Speed	Speed, S (mi/h)	Freq n	Freq (%)	Freq (%)	n*S	$n*S^2$
15	20	17.5	0	0.00%	0.00%	0.00	0.00
20	25	22.5	4	2.37%	2.37%	90.00	2,025.00
25	30	27.5	9	5.33%	7.69%	247.50	6,806.25
30	35	32.5	18	10.65%	18.34%	585.00	19,012.50
35	40	37.5	35	20.71%	39.05%	1,312.50	49,218.75
40	45	42.5	42	24.85%	63.91%	1,785.00	75,862.50
45	50	47.5	32	18.93%	82.84%	1,520.00	72,200.00
50	55	52.5	20	11.83%	94.67%	1,050.00	55,125.00
55	60	57.5	9	5.33%	100.00%	517.50	29,756.25
60	65	62.5	0	0.00%	100.00%	0.00	0.00
			169	**100.00%**		**7,107.50**	**310,006.25**

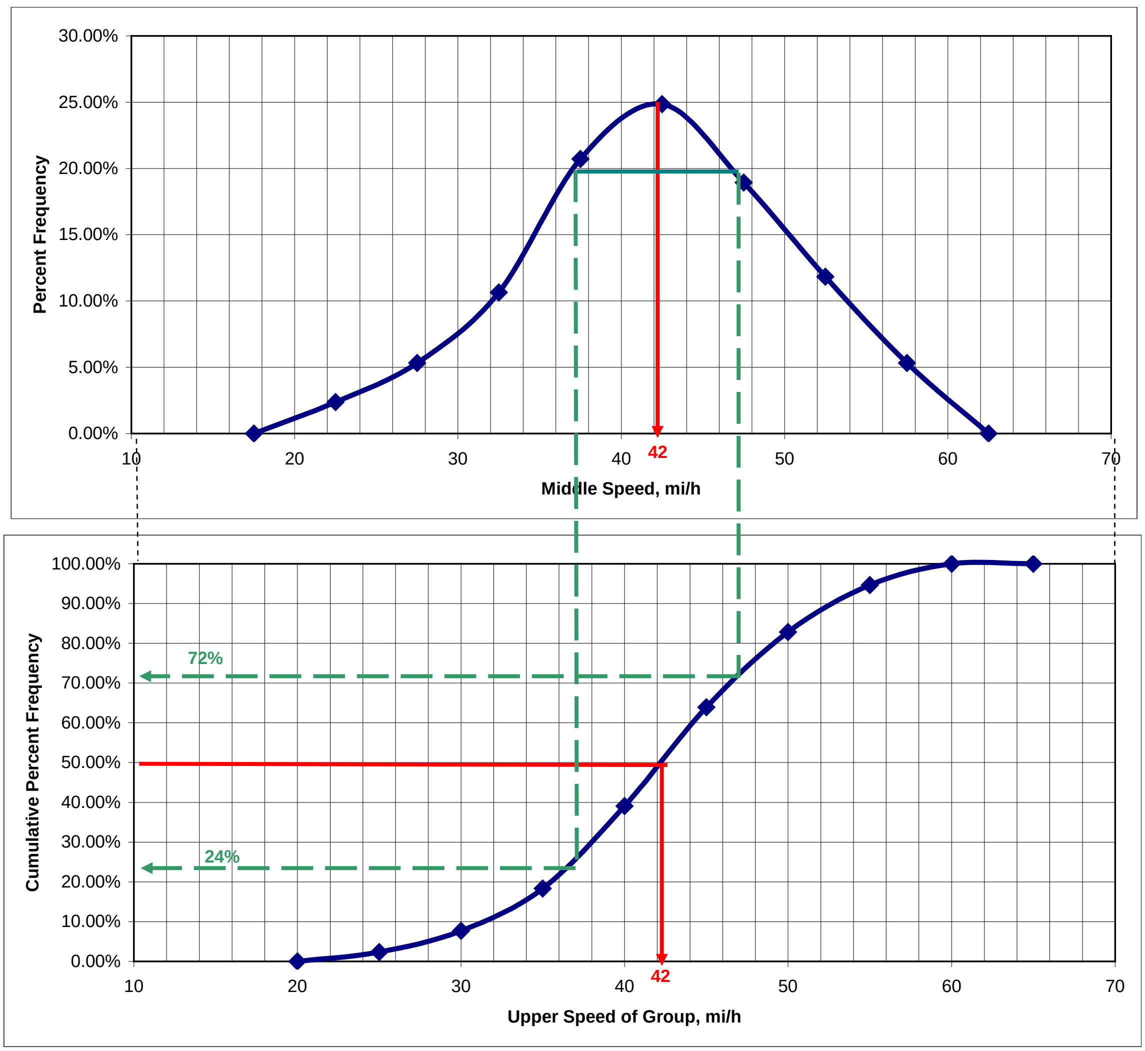


Figure 1: Frequency and Cumulative Frequency Distribution Curves

Part c:

The mean speed of the distribution is computed using Equation 11-3; the standard deviation is computed using Equation 11-5. Both use column totals from the Frequency Distribution Table.

$$\bar{x} = \frac{\sum_i n_i S_i}{N} = \frac{7107.50}{169} = 42.06 \ mi/h$$

$$s = \sqrt{\frac{\sum n_i S_i^2 - N\overline{x^2}}{N-1}} = \sqrt{\frac{310{,}006.25 - 169 * 42.06^2}{169-1}} = \sqrt{\frac{11{,}037.88}{168}} = \sqrt{65.7} = 8.11 \ mi/h$$

Part d:

The standard error of the mean, E, is computed as:

$$E = \frac{s}{\sqrt{N}} = \frac{8.11}{\sqrt{169}} = \frac{8.11}{13} = 0.624$$

Then:

$$95\% \ Confidence: \quad \mu = \bar{x} \pm 1.96E = 42.06 \pm (1.96 * 0.624) = 42.06 \pm 1.22 \quad (40.84 \ to \ 43.28 \ mi/h)$$

$$99.7\% \ Confidence: \ \mu = \bar{x} \pm 3E = 42.06 \pm (3 * 0.624) = 42.06 \pm 1.87 \quad (40.19 \ to \ 43.93 \ mi/h)$$

Part e:

Sample size is estimated using Equation 11-10 for 95% confidence:

$$n = \frac{3.84 s^2}{e} = \frac{3.84 * 8.11}{0.8} = 38.9, \ say \ 40 \ samples$$

Part f:

This question is answered by conducting a Chi-Squared Goodness of Fit test. In this test, actual observed frequency values are compared to theoretical frequencies that *would have been observed if the distribution were "Normal."* Table 2 shows the test computations.

Table 2: Chi-Square Goodness of Fit Test

Speed Group (mi/h)		Observed	Upper	Prob	Prob of	Theoretical	Combined Group		Group
High Speed	Low Speed	Freq n	Limit "z_d"	$z \le z_d$ Tab 7.3	of Being in Group	Freq f	f	n	χ^2
∞	60	0	∞	1.0000	0.0136	2.2984			
60	55	9	2.21208385	0.9864	0.0432	7.3008	9.5992	9	0.0374
55	50	20	1.59556104	0.9432	0.1354	22.8826	22.8826	20	0.3631
50	45	32	0.86806412	0.8078	0.1672	28.2568	28.2568	32	0.4959
45	40	42	0.36251541	0.6406	0.2393	40.4417	40.4417	42	0.0600
40	35	35	-0.2540074	0.4013	0.2091	35.3379	35.3379	35	0.0032
35	30	18	-0.8705302	0.1922	0.1241	20.9729	20.9729	18	0.4214
30	25	9	-1.487053	0.0681	0.0493	8.3317	11.5089	13	0.1932
25	20	4	-2.1035758	0.0188	0.0155	2.6195			
20	15	0	-2.7200986	0.0033	0.0033	0.5577			
TOTAL		**169**			**1.0000**	**169**	**169**	**169**	**1.5743**

After all speed groups are combined to insure that all values of f ≥ 5, the result is a Chi-Square value of 1.5743, with 7 – 3 = 4 degrees of freedom. From text Table 9-7, the probability of a value this high or higher is between 0.75 (for $X^2 = 1.923$) and 0.90 (for $X^2 = 1.064$). Interpolating:

X_d^2	Prob ($X^2 \ge X_d^2$)
1.064	0.90
1.574	?
1.923	0.75

$$? = 0.75 + (0.90 - 0.75) * \left(\frac{1.923 - 1.574}{1.923 - 1.064}\right) = 0.75 + (0.15 * 0.406) = 0.811 \ or \ 81.1\%$$

In order to *reject* the hypothesis that the data and the Normal Distribution are the same, the Prob ($X^2 \ge X_d^2$) would have to be *less than 5%*. Therefore, the hypothesis is confirmed. The data may be considered to be normally distributed.

Problem 11-2

a. To determine whether or not the observed reduction in speeds was significant, a Normal Approximation Test must be conducted:

b.

$$s_y = \sqrt{\frac{s_1^2}{N_1} + \frac{s_s^2}{N_2}} = \sqrt{\frac{4.8^2}{120} + \frac{5.3^2}{108}} = \sqrt{0.192 + 0.260} = \sqrt{0.452} = 0.672$$

$$z_d = \frac{(\overline{x_1} - \overline{x_2}) - 0}{s_d} = \frac{43.5 - 40.8}{0.672} = \frac{2.7}{0.672} = 4.02$$

From text Table 11-2, the Prob ($z_d \le 4.02$) = 0.9999. Thus, the difference is highly significant.

c. The standard error of the mean for the after sample is:

$$E = s/\sqrt{N} = 5.3/\sqrt{108} = 5.3/10.39 = 0.515$$

Therefore, it is 95% probable that the true mean speed of the distribution lies between 40.8 + (1.96*0.515) and 40.8 – (1.96*0.515) or 39.8 to 41.8 mi/h. As the target speed of 40 mi/h is in this range, it may be considered to have been achieved.

Problem 11-3

Note: This solution assumes that there is *one* lane being observed.

Table 3: Summary of Queue Counts

Clock Time	Cycle Number	No. of Vehicles in Queue at:			
		+0 s	+15 s	+30 s	+45 s
9:00	1	3	4	2	4
9:01	2	1	2	3	3
9:02	3	4	3	3	4
9:03	4	2	3	3	4
9.04	5	0	1	2	3
9:05	6	2	1	1	2
9:06	7	4	3	3	3
9.07	8	5	5	6	4
9:08	9	2	3	4	3
9:09	10	0	3	2	2
9:10	11	1	2	3	1
9:11	12	1	0	1	0
9:12	13	2	2	1	2
9:13	14	2	3	2	2
9:14	15	4	3	3	3
Sum		**33**	**38**	**39**	**40**
Total for All Time Periods:					**150**

$$\sum V_{qi} = 150$$

Then:

$$T_Q = \left(I_s * \frac{\sum_i V_{qi}}{V_T} \right) * 0.90 = \left(15 * \frac{150}{435} \right) * 0.90 = 4.66 \ s/veh$$

$$V_{SLC} = \frac{V_{STOP}}{N_c * N_L} = \frac{305}{15*1} = 20.3 \ vehs/cycle$$

$$FVS = \frac{V_{STOP}}{V_T} = \frac{305}{435} = 0.701$$

$$d = T_Q + (FVS * CF)$$

Where: *CF* = +2 (Text Table 11-11, FFS = 35 mi/h, V_{SLC} = 20.3)

Then: $d = 4.66 + (0.701*2) = 6.062 \ s/veh$

Problem 11-4

For 95% confidence:

$$n = \frac{3.84\,s^2}{e^2}$$

Table 4 executes this equation for tolerance levels (e) of 2 min, 5 min, and 10 min with base standard deviations (s) of 5 min, 10 min, and 15 min.

Table 4: Required Number of Samples

Tolerance (min)	Standard Deviations (min)		
	5	**10**	**15**
2	24	96	216
5	4	15	35
10	1	4	9

Problem 11-5

Table 5 uses the problem data to determine the average travel time and average running time in each section shown. These values are used to compute the average travel speed and the average running speed.

Note that data for the 1st segment must be added as follows: Cumulative Section Length = 0.50 mi; Cumulative Travel Time = 1.0 min; Delay = 0 s; No. of Stops = 0.

Then, using the data, Segment Lengths must be established, as well as travel times in each segment (converted to seconds). As an example, select the segment between checkpoints 3 and 4:

Section Length (mi) = 3.50 – 2.25 = 1.25 mi

Travel Time = 7 min, 30 s – 4 min, 50 s = 450 s – 290 s = 160 s.

Running Time = 160 – 25 = 135 s.

Average Travel Speed = (1.25/160)*3600 = 28.1 mi/h.

Average Running Speed = (1.25/135)*3600 = 33.3 mi/h

Table 5: Average Travel Speed and Average Running Speed

Section	Length (mi)	Travel Time (s)	Running Time (s)	Travel Speed (mi/h)	Running Speed (mi/h)
1	0.50	60	60	30.0	30.0
2	0.50	65	55	27.7	32.7
3	1.25	165	135	27.3	33.3
4	1.25	160	135	28.1	33.3
5	0.50	100	58	18.0	31.0
6	0.25	77	30	11.7	30.0
7	0.75	87	73	31.0	37.0

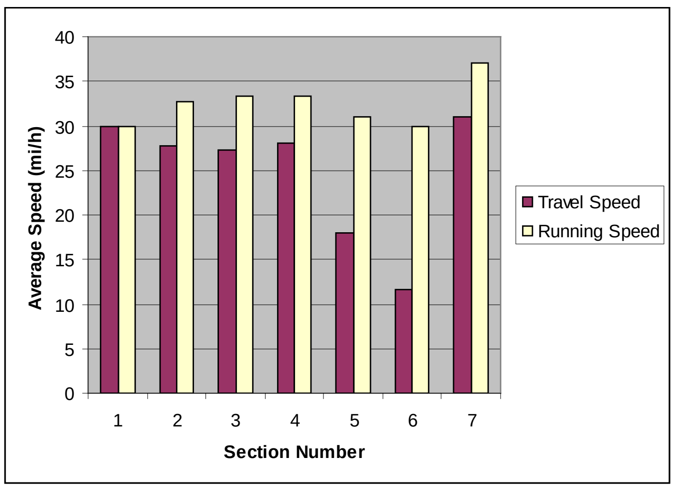


Figure 2: Average Travel Speed and Average Running Speed

The answer to Part b depends upon *which* measure is to have a tolerance of 3 mi/h – average travel speed or average running speed? We also need an estimate of the standard deviation of travel speed and/or running speed. The standard deviations are computed in Table 6 as:

$$s = \sqrt{\frac{\sum (x_i - \bar{x})^2}{N - 1}}$$

Table 6: Computation of Standard Deviations

Segment	Travel Speed (mi/h)	Running Speed (mi/h)	$(TS\text{-}Mean)^2$	$(RS\text{-}Mean)^2$
1	30	30	26.72	6.19
2	27.7	32.7	8.19	0.06
3	27.3	33.3	5.96	0.71
4	28.1	33.3	10.85	0.71
5	18.0	31.0	46.65	2.11
6	11.7	30.0	172.71	6.19
7	31.0	37.0	38.49	20.24
Total	**173.8**	**227.4**		
Mean	**24.8**	**32.5**		
STD			6.42	3.37

Then:

$$n = \frac{3.84\, s^2}{e^2}$$

$$n_{ATS} = \frac{3.84 * 6.42^2}{3^2} = 17.6, \ \ say \ \ 18 \ runs$$

$$n_{ARS} = \frac{3.84 * 3.37^2}{3^2} = 4.8, \ say \ 5 \ runs$$

If each car makes 5 runs, 3 cars would be needed to estimate ATS; only 1 would be needed to estimate ARS.

Solutions to Problems in Chapter 12

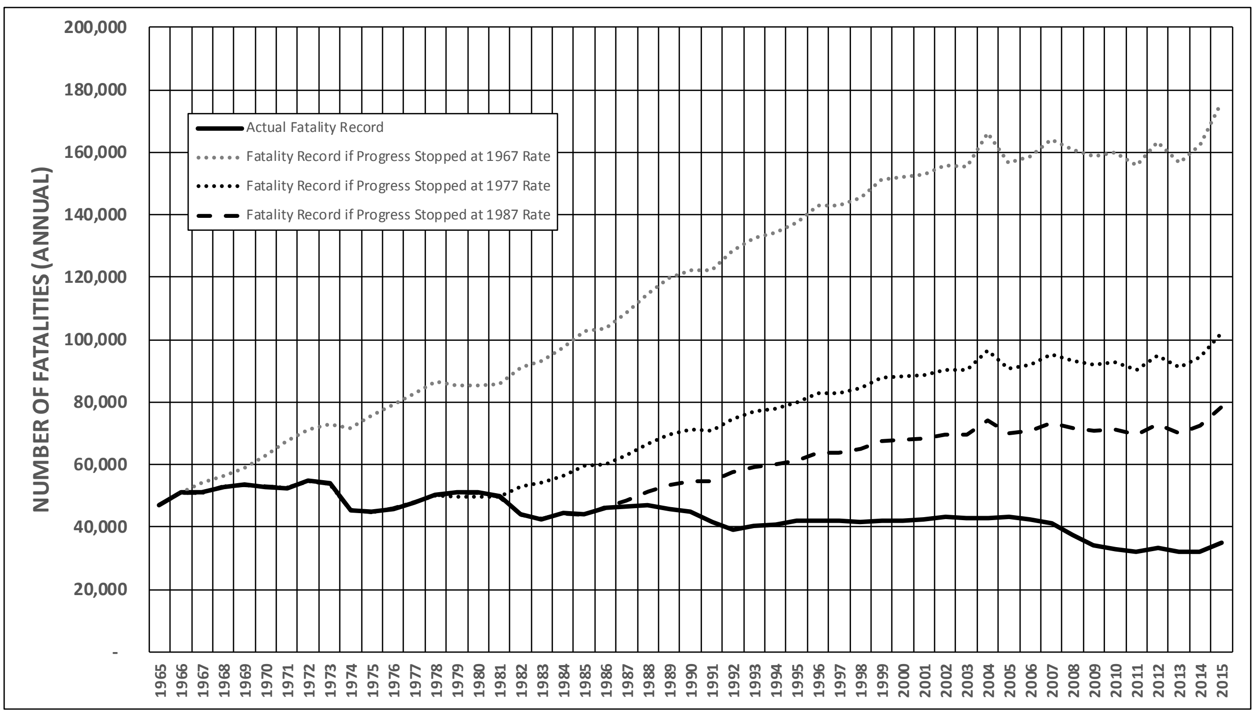
Highway Traffic Safety – An Overview

Problem 12-1

A larger scale version of Figure 12-2 is attached at the end of this solution set, marked simply as "Figure 2". Rather than go to source documents for the exact information, we chose to read the graph and construct the Table 12-1 on the next page:

- The first three columns show the year, fatality *rate* (per 100 million VMT), and actual fatality record (annual number);
- The next three columns show what would have happened if the progress in decreasing the fatality rate had stopped at the rates observed in 1967, 1977, and 1987 and scaling the number of fatalities up by a factor of {hypothetical stopped progress rate / actual rate from the 2nd column}.

The result is spectacular:



A response from students, or anyone, that "But this would never be allowed to happen!" would be very understandable. Without the improvements shown in the rates, the number of fatalities would be 400% higher (at the 1966 rate) or 190% higher (at the 1976 rate) or 123% higher (at the 1986 rate).

It is "almost obvious" that if the rates had climbed as they could have, aggressive action would have been mandated. VMT trends could have led to the above.

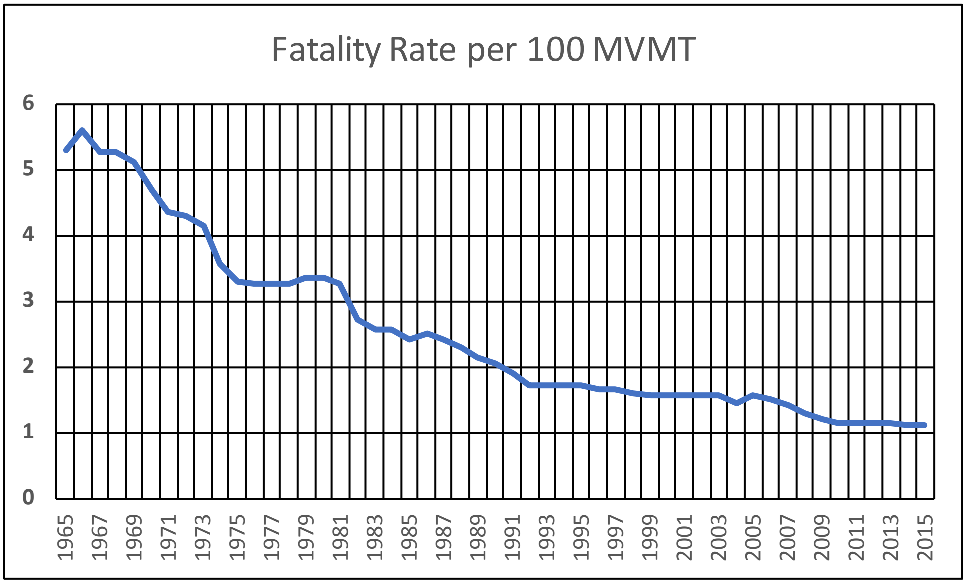
Retrospectively, the number of fatalities in the 50,000 range circa 1965 did lead to a great emphasis on improved safety in vehicle design, infrastructure, and other elements. The above curves are simply indicative of what could have happened if change did not occur, or if it were not so persistent over the decades. So, it should be a tribute to those involved that we have a less grim present.

Table 12-1, for Solution re Problem 12-1

	From Figure 12-2		Fatality Rates		
			5.60	3.25	2.50
	Fatality Rate per 100 MVMT	Actual Fatality Record	Fatality Record if Progress Stopped at	Fatality Record if Progress Stopped at	Fatality Record if Progress Stopped at
1965	5.3	47,089	47,089	47,089	47,089
1966	5.6	51,000	51,000	51,000	51,000
1967	5.25	51,000	54,400	51,000	51,000
1968	5.25	52,800	56,320	52,800	52,800
1969	5.1	53,500	58,745	53,500	53,500
1970	4.7	52,800	62,911	52,800	52,800
1971	4.35	52,400	67,457	52,400	52,400
1972	4.3	54,800	71,367	54,800	54,800
1973	4.15	54,000	72,867	54,000	54,000
1974	3.55	45,500	71,775	45,500	45,500
1975	3.3	44,700	75,855	44,700	44,700
1976	3.25	45,800	78,917	45,800	45,800
1977	3.25	48,000	82,708	48,000	48,000
1978	3.25	50,200	86,498	50,200	50,200
1979	3.35	51,000	85,254	49,478	51,000
1980	3.35	51,000	85,254	49,478	51,000
1981	3.25	49,700	85,637	49,700	49,700
1982	2.7	44,000	91,259	52,963	44,000
1983	2.55	42,500	93,333	54,167	42,500
1984	2.55	44,300	97,286	56,461	44,300
1985	2.4	44,000	102,667	59,583	44,000
1986	2.5	46,200	103,488	60,060	46,200
1987	2.4	46,500	108,500	62,969	48,438
1988	2.3	47,100	114,678	66,554	51,196
1989	2.15	45,900	119,553	69,384	53,372
1990	2.05	44,800	122,380	71,024	54,634
1991	1.9	41,500	122,316	70,987	54,605
1992	1.7	39,000	128,471	74,559	57,353
1993	1.7	40,200	132,424	76,853	59,118
1994	1.7	40,800	134,400	78,000	60,000
1995	1.7	41,800	137,694	79,912	61,471
1996	1.65	42,100	142,885	82,924	63,788
1997	1.65	42,100	142,885	82,924	63,788
1998	1.6	41,500	145,250	84,297	64,844
1999	1.55	41,800	151,019	87,645	67,419
2000	1.55	42,100	152,103	88,274	67,903
2001	1.55	42,300	152,826	88,694	68,226
2002	1.55	43,100	155,716	90,371	69,516
2003	1.55	43,000	155,355	90,161	69,355
2004	1.45	43,000	166,069	96,379	74,138
2005	1.55	43,300	156,439	90,790	69,839
2006	1.5	42,500	158,667	92,083	70,833
2007	1.4	41,000	164,000	95,179	73,214
2008	1.3	37,300	160,677	93,250	71,731
2009	1.2	34,000	158,667	92,083	70,833
2010	1.15	32,800	159,722	92,696	71,304
2011	1.15	32,000	155,826	90,435	69,565
2012	1.15	33,500	163,130	94,674	72,826
2013	1.15	32,200	156,800	91,000	70,000
2014	1.1	31,900	162,400	94,250	72,500
2015	1.12	35,092	175,460	101,829	78,330

% GREATER THAN ACTUAL	400%	190%	123%
FACTOR, USING ACTUAL AS BASELINE	5.00	2.90	2.23

The student can continue the exercise, for "what if progress had stopped at the rates for" 1996 and 2006. Clearly, from Figure 12-2 of the text, there were major gains over several decades, with further improvements beyond 1992 – while still impressive (1.7 to approximately 1.1, a 35% decrease in rate) – saw years of plateauing:



Problem 12-2

In the year the 5th edition of the textbook was being prepared, there was an "uptick" of some 10% in the number of fatalities. Speculation immediately focused on distracted driving, which had been receiving much attention due to texting and use of smartphones --- by drivers and pedestrians, both.

We cannot offer a "right answer" for this assigned problem, because more data and more analysis is needed. Is the underlying root cause:

a) Distracted driving, as some postulated, or
b) A simple increase in VMT from 2014 to 2015[1], as the economy improved (note that the fatality *rate* did not substantially change in 2010 to 2015; see the chart immediately above), or
c) A random fluctuation with no causality, or
d) A combination of these factors?

The authors note that 10% is indeed an unusual bounce in this data, but the VMT question needs to be answered --- the fatality rate did not change. Distracted driving is a reality and a real problem. But that labeling can be speculation rather than analysis. The student

[1] This can be fact-checked by the student, as to whether there was a 10% increase in VMT in the same period.

assigned this problem will have better sources of data, from later years, and access to analyses not yet done.

<u>Problem 12-3</u>

Figure 12-4 is reproduced below for convenience. Clearly, the percentages for the single vehicle case add to substantially more than 100%, as do the percentages for the multiple vehicle case.

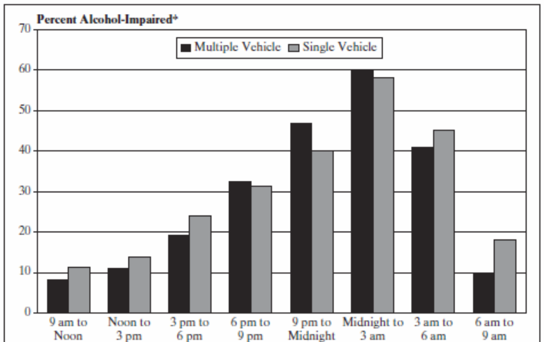


Figure 12.4: Percent of Persons Killed in Alcohol-Impaired Driving Crashes By Time of Day
(*Source: Traffic Safety Facts 2015*, National Highway Traffic Safety Administration, U.S. Department of Transportation, Washington, D.C., 2017, Pg 111.)

So, it seems clear that these are not the distributions of persons killed in alcohol-impaired driving crashes by time of day.

Let us focus on the single vehicle case from midnight to 3am: about 58% <u>of persons killed in this time period</u> were part of a crash that was designated as an alcohol-impaired event.

> Think of it this way, even if it sounds like an over-simplification: traffic is light during those hours, almost everywhere; the bars are emptying, parties are ending, and people are going home; given the concentration of such people in the midnight to 3am time slot, and given the (increased) probability of a crash when alcohol is involved, is it a surprise that 58% of the fatalities are in alcohol-related crashes, and (only) 42% in crashes that do not involve alcohol?
>
> Note that this does not say the dead person was necessarily impaired.
>
> Look at it as conditional probabilities:
>
> - o Given the time slot, is it more likely that alcohol is involved in any random vehicle on the road at that time?
> - o Given alcohol involved, isn't it true that the probability of a crash has increased?
> - o Given such a crash, is it more likely or less likely that a fatality will occur (vehicle speed, etc.)?

Actually, the last point can be answered “no” and the stage is still set for a 58%-42% split of which types of accidents that fatalities are concentrated in.

So, is Figure 12.4 now moreso a profile of when people are drinking? Add to that a greater propensity for their vehicles be involved in accidents (this should be easy to fact-check) and have more serious accidents, Figure 12.4 then becomes the drinking profile skewed by the frequency and severity of crashes involving that segment of the population?

The authors believe that this line of reasoning is quite adequate for responding to Part “a” of the problem statement.

Indeed, it essentially addresses Part “b” also, except for parsing the underlying data in different ways. But the data is in the FARS (FARS = Fatality Analysis Reporting System[2], USDOT/NHTSA) data base and in reports based upon it. The data is easily parsed to show crash fatalities by time of day, fatality rates by time of day (with information on VMT by time of day retrieved), and so forth.

Problem 12-4

A web search on “graduated licensing” results in a number of related topics on the list as one types, and even with those two words, provides an extensive set of references on definitions of the concept, state laws, assessments of effectiveness, and much more. The student has a rich set of material to address this problem.

The National Institutes of Health (NIH) funded a number of studies of graduated licensing, and reported in 2011 that

> “Programs that grant privileges to new drivers in phases — known as graduated licensing programs — dramatically reduce the rate of teen driver fatal crashes, according to three studies funded by the National Institutes of Health.
>
> “Such graduated licensing laws were adopted by all 50 states and the District of Columbia between 1996 and 2011. The NIH-supported research effort shows that such programs reduced the rate of fatal crashes among 16- 17-year-olds by 8 to 14 percent.
>
> “Reductions in fatal crashes were greatest in states that had enacted other restrictions on young drivers. The greatest reductions in young driver crashes were seen in states that had adopted graduated driver licensing laws in combination with mandatory seat belt laws or laws

[2] Once named the “Fatal Accident Reporting System”, but the word “crash” has replaced “accident” in our terminology: the word “accident” tends to have a connotation of something random and rather unavoidable; “crash” is a statement of fact without a connotation.

requiring a loss of the driver's license as a penalty for possession or use of alcohol by youth aged 20 or younger.

"In addition, limiting driving at night or with teenaged passengers, in combination with graduated licensing laws, had greater reductions in overall crash rates involving teen drivers than graduated licensing laws alone".

Refer to https://www.nih.gov/news-events/news-releases/graduated-drivers-licensing-programs-reduce-fatal-teen-crashes

Problem 12-5

Deaths and Accidents Per 100,000 Population:

$$Deaths/100{,}000\,Pop = 15*\left(\frac{100{,}000}{50{,}000}\right) = 30\ per\,100K\ Population$$

$$Acc/100{,}000\,Pop = 360*\left(\frac{100{,}000}{50{,}000}\right) = 720\ per\,100K\ Population$$

Deaths and Accidents Per 10,000 Registered Vehicles

$$Deaths/10{,}000\,\mathrm{Re}\,g\ Veh = 15*\left(\frac{10{,}000}{35{,}000}\right) = 4.28\ per\,10K\ \mathrm{Re}\,g\,Veh$$

$$Acc/10{,}000\,\mathrm{Re}\,g\ Veh = 360*\left(\frac{10{,}000}{35{,}000}\right) = 102.9\ per\,10K\ \mathrm{Re}\,g\,Veh$$

Deaths and Accidents Per 100,000,000 Vehicle-Miles Travelled

$$Deaths/100MVM = 15*\left(\frac{100{,}000{,}000}{12{,}000{,}000}\right) = 125\ per\,100MVM$$

$$Acc/100MVM = 360*\left(\frac{100{,}000{,}000}{12{,}000{,}000}\right) = 3{,}000\ per\,100MVM$$

National statistics were addressed in the text of the chapter. The rates in this locality are very high compared to national statistics, and merit a thorough investigation.

Problem 12-6

Observation	**Remedy to Consider**
Sideswipe and turning accidents may be due to the off-set intersection geometry, which has vehicle paths intersection at	Install a 3-phase signal with a fully protected LT phase for the NS street.

unexpected positions within the intersection.	
Some of the rear-end collisions may be related to signal visibility problems. This should be checked, as there are only two pole-mounted signal heads.	Use overhead signal heads on span wire.
Some sideswipe accidents may be related to vehicles approaching in the wrong lane for their movement.	Better lane-use control markings and signing.
Pedestrian accidents may be due to the awkward location of the crosswalks or related reasons.	Place Pedestrian Signals at proper locations; check signal timing for pedestrians.

The revised condition diagram can be constructed from the remedies cited in each of the above 4 observations.

Problem 12-7

The number of accidents at the 4 leg signalized intersection must be predicted in four categories: Multivehicle Crashes, Single-Vehicle Crashes, Vehicle-Pedestrian Crashes, and Vehicle-Bicycle Crashes. Note that separate predictions for crashes involving fatalities and/or injuries and property damage only in each category can be done, but are not necessary to answer the question. These are, therefore, not shown in this solution.

Equation 12-3 is used with Table 12.4 for multivehicle crashes:

$$N_{bmv} = \exp\left[a + b\ln(AADT_{maj}) + c\ln(AADT_{\min})\right]$$

Where: $AADT_{maj}$ = 60,000 (given)

$AADT_{min}$ = 25,000 (given)

a = -10.99 (Table 12.4, total)

b = 1.08 (Table 12.4, total)

c = 0.23 (Table 12.4, total)

Then:

$$N_{bmv} = \exp\left[-10.99 + 1.08\ln(60{,}000) + 0.23\ln(25{,}000)\right] = \exp\left[-10.99 + (11.882) + (2.329)\right] = \exp[3.221] = 25.05$$

Equation 12-6 and Table 12-4 are used for single-vehicle crashes:

$$N_{bsv} = \exp\left[a + b\ln(AADT_{maj}) + c\ln(AADT_{\min})\right]$$

Where: $AADT_{maj}$ and $AADT_{min}$ as above

a = -10.21 (Table 12.5, total)

b = 0.68 (Table 12.5, total)

c = 0.27 (Table 12.5, total)

Then:

$$N_{bsv} = \exp[-10.21 + 0.68\ln(60{,}000) + 0.27\ln(25{,}000)] = \exp[-10.21 + 7.4814 + 2.7342] = \exp[0.0056] = 1.00$$

Equation 12-7 and Table 12-6 are used for vehicle-pedestrian crashes.

$$N_{pedbase} = \exp\left[a + b\ln(AADT_{total}) + c\ln\left(\frac{AADT_{\min}}{AADT_{maj}}\right) + d\, n_{lanesxl}\right]$$

Where: $AADT_{min}$ and $AADT_{maj}$ as previously.

$AADT_{total}$ = 60,000+25,000 = 85,000

$n_{lanesxl}$ = 6 (given)

a = -9.53 (Table 12.6)

b = 0.40 (Table 12.6)

c = 0.26 (Table 12.6)

d = 0.04 (Table 12.6)

Then:

$$N_{pedbase} = \exp\left[-9.53 + 0.40\ln(85{,}000) + 0.26\ln\left(\frac{25{,}000}{60{,}000}\right) + 0.04 * 6\right]$$
$$N_{pedbase} = \exp[-9.53 + 4.540 - 0.228 + 0.240] = \exp[-4.978] = 0.60$$

The last category of accidents is vehicle-bicycle crashes. The prediction of the number of such accidents depends upon the predictions for multivehicle and single-vehicle crashes – AFTER any Crash Modification Factors have been applied. Thus, the next step is to apply all applicable CMFs to the predictions for multivehicle and single-vehicle crashes. These factors identified in Table 12.8, and found in Tables 12.9 and 12.10, along with several equations (12-9, 12-10, 12-11). The applicable CMFs are listed below, along with their source or computation.

CMF_{LT} = 0.66 (Table 12-9, 4 LT lanes)

CMF_{RT} = 0.92 (Table 12-9, 2 RT lanes)

CMF_{L} = 1 – (0.38*0.235) = 0.91 (Equation 12-9)

CMF_{SP} = 0.78 (Table 12.10, 4 protected LTs)

An additional CMF for the existence of red-light cameras requires knowledge of the proportion of multivehicle crashes that are right-angle (p_{ra}) and rear-end (p_{re}). These must be known, but are not specified for the problem. This adjustment would only apply to multivehicle accidents. Since the information is not given, we will assume a value of 1.00 for this adjustment. There are no prohibitions on RTOR, so the CMF for this condition is 1.00 by definition (it is the base condition). Predicted multivehicle and single-vehicle accidents may now be adjusted:

$$N_{pred,mv} = 25.05 * 0.66 * 0.92 * 0.91 * 0.78 = 10.80$$
$$N_{pred,sv} = 1.00 * 0.66 * 0.92 * 0.91 * 0.78 = 0.43$$

Crash modification factors (CMF) also apply to pedestrian-vehicle crashes. These are found in Table 12.11:

CMF_{BS} = 1.00 (Table 12.11, no bus stops)

CMF_{SCH} = 1.35 (Table 12.11, 1 school within 1,000 ft)

CMF_{ALC} = 1.12 (Table 12.11, 2 liquor stores within 1,000 ft)

Then:

$$N_{pred,ped} = 0.60 * 1.00 * 1.35 * 1.12 = 0.91$$

The number of bicycle-vehicle crashes may now be estimated using Equation 12-8:

$$N_{pred,bike} = (N_{pred,mv} * N_{pred,sv})\ f_{bike}$$

Where: $N_{pred,mv}$ = 10.80 (computed above)

$N_{pred,sv}$ = 0.43 (computed above)

f_{bike} = 0.015 for all 4SG intersections

Then:

$$N_{pred,bike} = (10.80 + 0.43) * 0.015 = 0.17$$

The total number of crashes per year can now be estimated using Equation 12-1, which can be expressed as:

$$N_{pred,\text{int}} = c_i \left(N_{pred,mv} + N_{pred,sv} + N_{pred,ped} + N_{pred,bike} \right)$$

Where the calibration coefficient (c_i) is given as 1.04, and all other values are as previously computed. Then:

$$N_{pred,\text{int}} = 1.04 * [10.80 + 0.43 + 0.91 + 0.17] = 12.80\ crashes / yr$$

The *HSM* predicts that this very busy intersection will experience 12.80 crashes per year, the vast majority of which will be multivehicle crashes. While not asked for in this problem, the number of fatal and injury accidents could have been separately predicted to obtain a general indication of crash severity at this location.

Solutions to Problems in Chapter 13

Parking – Characteristics, Studies, Programs, and Design

Problem 13-1
From text Table 13-1, for a high-rise apartment complex without significant transit access, the expected peak parking demand is given by:

$$P = 1.04X + 130$$

where X is the number of dwelling units. Thus:

$$P = (1.04 * 600) + 130 = 754 \ \textit{parking spaces}$$

Problem 13-2
From text Table 13-5, for a shopping center with 600,000 ft^2 of gross leasable area (GLA), with 10% of the space occupied by theaters and restaurants, the parking ratio is expected to be 4.5 spaces/1,000 ft^2 GLA. Thus:

$$P = 4.5 * 600 = 2{,}700 \ \textit{parking spaces}$$

Problem 13-3
Text Table 13-6 gives typical zoning regulations for parking. The categories, however, are not entirely consistent with Tables 13-1 and 13-5, used to estimate parking demand for the situations described in Problems 13-1 and 13-2.

Three zoning criteria are given for "multi-unit dwellings:" 1.25 per dwelling unit for studio apartments, 1.50 per dwelling unit for one-bedroom apartments, and 2.00 per dwelling unit for two- or more bedrooms per apartment. The details are not given in Problem 13-1, so an assumption must be made. If the building is a mix of all of the above, then the rate for a one-bedroom apartment should be sufficient. Thus, zoning would require, for Problem 13-1, 1.50*600 or 900 parking spaces. This is more than the estimated peak parking demand, demonstrating the difficulty in using national averages based upon different studies in setting zoning requirements.

For the shopping center of Problem 13-2, the recommended zoning requirement is shown in text Table 13-7. For the shopping center as described, the recommended zoning requirement is 4.5 spaces per 1,000 ft^2 of GLA. This is the same as the estimate of parking demand, or 2,700 parking spaces.

Problem 13-4
With the information given, the number of parking spaces needed is best estimated using Equation 13-1:

$$P = \frac{N * K * R * A * pr}{O}$$

where: N = *2,000* employees
K = *0.85* of trips expected in peak hour
R = *1.00* person-destinations per day per employee
A = *0.93* of trips arrive by automobile
pr = *1.00* of trips have primary destination at the location
O = *1.3* persons/auto

Then:

$$P = \frac{2{,}000*0.85*1.00*0.93*1.00}{1.3} = 1{,}216 \ \ parking \ \ spaces$$

<u>Problem 13-5</u>

Equation 13-2 is used to estimate the number of vehicles that may be parked in the 14-hour time period described in the study:

$$P = \left(\frac{\sum_n NT}{D} \right) * F$$

Then:

$$P = \frac{(100*14)+(150*8)+(200*6)+(300*10)}{35/60} * 0.90 = \frac{6{,}800}{0.583} * 90 = 10{,}497 \ \ parked \ \ vehicles$$

Obviously, not all of these would be parked at one time, given the short average duration of 35 minutes (0.583 hrs).

<u>Problem 13-6</u>

The parking study sheet included in the problem statement has all of the information needed to solve this problem. Parking totals for each time period are summed (each column). For each parking space (row), the number of vehicles parked for one, two, three, four, etc 30-minute parking periods is noted.

A vehicle noted as parked in one ½ hour interval is assumed to be parked for the full ½ hour.

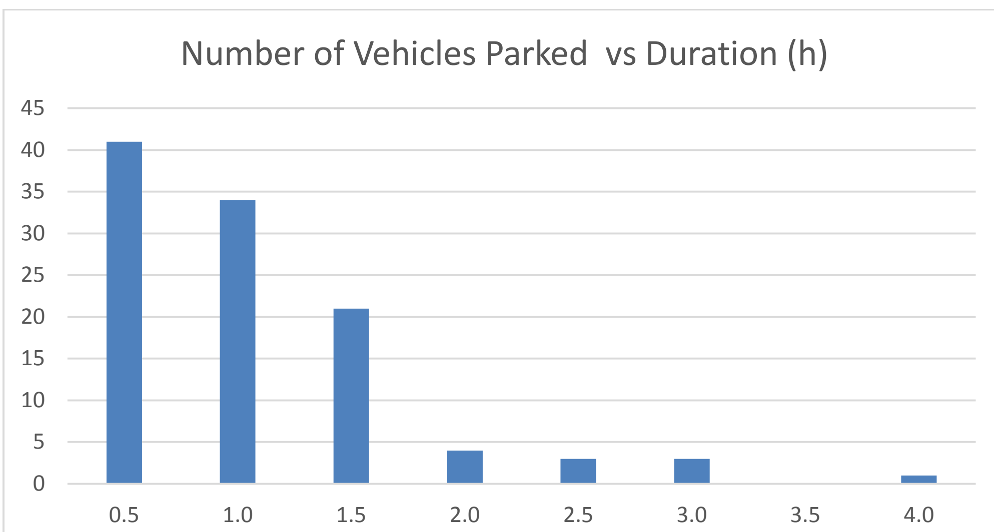
																Number of Periods Parked							
Parking Space	**7:00**	**7:30**	**8:00**	**8:30**	**9:00**	**9:30**	**10:00**	**10:30**	**11:00**	**11:30**	**12:00**	**12:30**	**1:00**	**1:30**	**2:00**	**1 Per**	**2 Per**	**3 Per**	**4 Per**	**5 Per**	**6 Per**	**7 Per**	**8 Per**
1 hr meter	100	√		150	√	√	246	385		691	√	√		810	√	2	2	2					
1 hr		468	√	630	√	485		711	888	927	√	√	108	√		4	4						
1hr	848	911	√	√	221	747	922	√		787	√	452	√		289	5	3	1					
1 hr			206	√	242	√	√		899	√	205	603	812	√	√	2	2	2					
1 hr			566	665	√	333	848	√	999		720		802	√		4	3						
1 hr		690		551	√	√	347	√	265	835	486	√		721	855	5	2	1					
Hydrant								777								1							
2 hr meter			940	√	√	505	608	√	√	√	121	123	√		880	3	1	1	1				
2 hr	636	√	√	√	√		582	√	√	811	919	√	711	√	√	1	1	2		1			
2 hr		399	√	√	401	904	√	√	789	√	556	√	√	√	232	2	1	1	1				
2 hr		416	√	√	√	√	√		658	√	292	844	493	√	√	2	1	1			1		
2 hr	188	√	√		655	558	√	√	√	213	√		779	√	√	1	1	2	1				
2 hr				277	√	336	409	√	√	884	√	√	713	895	431	4	1	2					
2 hr			837	√	√	418	575	√	952	√	√	√	√		762	2	1	1		1			
2 hr		506	√	√		786	√	√	√	527	606	√	385	√	√	1	1	2	1				
Hydrant						518				758						2							
3 hr meter		079	√	√	√	√	√	√	√		441	√	611	√	√		1	1					1
3 hr	256	√	√	√	√		295	√	√	338	√		499	√	√		1	2		1			
3 hr			848	√	√	√	√	√		933	√	√	√	√	√						2		
Bus Stop						740	142										2						
Bus Stop						915											1						
Bus Stop																							
Bus Stop															818		1						
Bus Stop				888		175	755								397		4						
Total Parkers	**5**	**11**	**14**	**16**	**16**	**19**	**18**	**16**	**14**	**16**	**17**	**14**	**15**	**15**	**15**	**41**	**34**	**21**	**4**	**3**	**3**	**0**	**1**
Legal Spaces:	**17**																						

Legal Parking Space

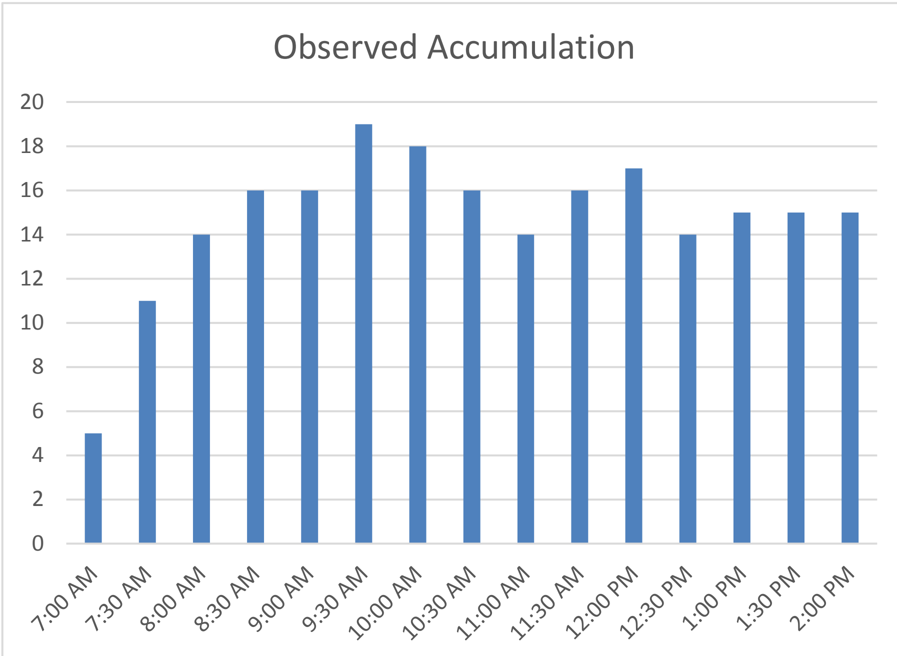
Illegally Parked Vehicle

Note that there are 17 legal parking spaces, and that 11 illegal parkers have been identified. The sum of data columns (shown in red) is 221 – meaning that 221 vehicles parked for ½ hour have been noted. Many are the same vehicle parked for more than one ½ hour observation period. The total number of vehicles observed is the sum of the vehicles in "number of periods parked" columns – or 76 vehicles.

(a) The duration distribution is given by the sums of the "number periods parked" columns of the table, with each period indicating ½ hour of parking. The distribution is shown as a bar chart below.



(b) The accumulation pattern is defined by the column totals for each ½ hour period observed. It is shown in the figure below.



(c) 76 vehicles were observed parked for 221 ½-hour periods. The average parking duration is therefore:

$$D = \frac{221*30}{76} = 87.2 \text{ min} = 1.45\ h$$

(d) 11 out of 76 parkers were in illegal spaces. The parking violation rate was:

$$PVR = \frac{11}{76} = 0.145\ of\ 14.5\%$$

To obtain the overtime rate, the number of parkers exceeding the maximum limit of their meters would have to be observed. The table contains this information. For a 1 hr meter, any vehicle remaining for 3 or more time periods is overtime; for a 2 hr meter, any vehicle remaining for 5 or more time periods is overtime; for a 3 hr meter, any vehicle remaining for 7 or more time periods is overtime.

For 1 hr meters, there are 6 overtime parkers.
For 2 hr meters, there are 3 overtime parkers.
For 3 hr meters, there is 1 overtime parker.

The overflow rate is, therefore:

$$OR = \frac{10}{76} = 0.131 \; or \; 13.1\%$$

(e) The parking turnover rate is given by Equation 13-4:
(f)

$$TR = \frac{N_T}{P_S \, T_S}$$

where: N_T = number of observed parkers (76)
P_S = number of legal parking stalls (17)
T_S = duration of parking study, h, (7)

Then:

$$TR = \frac{76}{17 * 7} = 0.263 \; vehicle / h / stall$$

There is obviously a supply problem here, given that the maximum accumulation of 19 vehicles exceeds the number of legal spaces (17), and that the violation and overtime rates are significant.

Solutions to Problems in Chapter 14

Traffic Impact Studies and Analyses

Preface

This one chapter (supplemented by the required purchase of Reference [6]) has been used as the basis for an entire 3-credit course by one of the authors, so the instructor must temper his/her expectations of the class with regard to this chapter and its problems. Full use of this problem set, with Reference [6] as a support document plus one small project on writing the proposal for (and cost estimating) a modest traffic study proved to be an intense experience for the students involved, and served as a capstone design course.

But every student should have some exposure to the subject of traffic impact due to development, and to related mitigation efforts.

At the same time, use of this chapter has to be tailored to both (1) the instructor's actual knowledge and experience with traffic impact studies, and (2) the student's command of such tools as Synchro & SimTraffic, VISSIM or AIMSUN, and/or TruTraffic. The authors express no preference amongst these tools, and have had them all used on projects in professional practice. Above all, the authors remind both instructor and students that it is the local jurisdiction that will likely specify the tool that must be used.

If one were going to limit attention to two lectures on traffic impact in a course (due to competing needs), the recommendation is that

- Sections 14.1 to 14.3 be gone over, in overview form;
- Section 14.4 (Case Study 1) be treated by indicating the problem as stated, focusing on Figure 14.3 and then Figure 14.4, reducing the problem at hand to simply an issue of where to locate the driveway so that it does not mess up the time-space diagrams in Figure 14.4 too much (its placement should allow it to be serviced in the blank parts of the time-space diagram, when main street traffic is not flowing);
- In preparing for Case Study 2, alert the students to 3 key points in this abbreviated treatment:
    - Any traffic impact analysis will look at and compare three or four traffic conditions:
        - The existing condition;
        - The future no-build, which assumes normal growth but without the project at hand;
        - The future build, which assumes both normal growth and the project at hand being implemented;

        - The future build, with mitigation to meet local requirements, to the extent feasible;
    - The key comparison is almost always the future build, compared to the future no-build. The baseline for comparison is the future no build, <u>not</u> the existing condition. The impact assessment should be an "apples-to-apples" comparison *in the relevant future year*;
    - While it is highly desirable to fully mitigate any impact (and it leads to easier approvals), it is not generally a legal requirement that there be "no impact". Rather than a requirement for "no impact" after mitigation, the true requirement of the environmental laws is that the decision-maker (e.g. transportation commissioner) be full informed when the decision is rendered, even if there is a negative impact.
        - Challenges arise out of representations of incomplete analysis or things not studied; they do not arise out of the decision-maker's right to decide, but rather whether they were fully informed at the time;
        - Of course, the process does generally move more smoothly if impacts are fully mitigated.
- Figure 14.5 be used as the basis for "talking through" the steps in the process, applied to Case Study 2. Table 14.4 be used to illustrate some local requirements adapted from an actual local ordinance. Note that considering the property as two distinct projects requires internal buffer zones between them, and can inhibit internal circulation;
- If time permits, assign <u>one</u> intersection for analysis: existing condition, future no build, future build.

If one is covering Chapter 14 in only one lecture, then the above can be treated in less detail and the actual problem solution can be skipped.

Note that Case Study 2 has twenty "Discussion Points", shown in bold italicized titles on paragraphs in the text. The purpose is to guide the work when Case Study 2 is done in its entirety, and to provide insights for the students and in some cases for the instructor.

Again, a word of caution: unless more than two lectures are devoted to this chapter, it may be totally unrealistic to assign <u>any</u> problems from the end of this Chapter. They are almost all of the variety of "execute and submit" the case study work.

So, the overview is very valuable to someone entering the field, but the actual work is quite time-intensive. Students should at least be exposed to the jargon used and the issues raised in this chapter.

"Solution" material to follow is more in the spirit of guidance to be provided by the instructor to the students.

The authors have found that these problems are best addressed by groups of students, generally 3 per group but with 4 per group in larger classes.

Problem 14-1

This problem requires the use of Synchro for the signal optimization and assumes SIMTraffic for the visualization, although the text also allows the use of VISSIM or AIMSUN (local availability of tools will determine which is used, in this student exercise).

The comment in the text is that the metrics produced by the simulation model may differ from that produced by Synchro. Historically, this has been true *even if the simulation model used is SIMTraffic, which many people have thought is the same model as Synchro.*

As commented above, the best solution is likely the one that disrupts the time-space rendering (Figure 14.4) the least. But the simulation models handle queueing differently than Synchro, so one may encounter some different results. Which rendering is better? In general, the authors have preferred the simulator output over the Synchro output. In some short course work, one of the authors helped produce cases that showed radical differences at the time between even Synchro and SIMTraffic, because of their imbedded mechanisms and assumptions and defaults.

In using the simulation models, one does have to be aware of the default settings --- some may not be valid for the local area. Think of discharge headways and lane merging rules as prime examples. So, using the simulation boxes "out of the box" without reviewing the defaults is not good practice.

Problem 14-2

The two prime measures to look at are arterial travel time and delay by approach for each link on the arterial. Both lead to assessments of the level of service.

The mitigation may be as simple as improved signal timing and coordination. Given the magnitude of the SB volumes, this might well not be the case.

Given 100 or 200 vph added to each of the SB and NR flows that turn into the driveway and a like number exiting the driveway, a few immediate questions come to mind:

- Is a signal required at the driveway? Given the volumes specified and the existing traffic, the answer is probably "yes" but the traffic signal warrants must be checked;
- How many lanes are needed in/out on the driveway? Is a Main Street NB right turn lane needed at the driveway?
- Will the left turn bays on Main Street suffice for the new left turning volume into the development, from the SB flow?

- Can one SB left turning lane suffice? At 200 vph, it is about 3.3 vehicles per minute. Allowing for a cycle length of 90 seconds, think 5 vehicles per cycle length. If one allowed for random arrivals during a given cycle, this could mean up to 10 vehicles of storage.
- Given how heavy the SB thru traffic is in Figure 14.3 and adding 100 or 200 vph due to the development (part of the SB flow north of Avenue B, then turning into the new driveway, but with the new volume levels kept due to exiting traffic), do the existing number of lanes suffice? Even with small cross street volumes, it may well be that a third SB thru lane is needed along the entire arterial;
- The initial wisdom of "fit the driveway traffic into the gaps shown in Figure 14.4" may be totally superceded by the need to assure that traffic at the new signal does not back up across Avenue B --- so placing the driveway further south may work best (but then, SB queues at Avenue C may contra-indicate this, particularly when part of that queue comes from the driveway;
- Multi-phase operation has to be considered, at least at the driveway. And for the purposes of coordinated signal operation, the driveway need may dictate the cycle length for the entire arterial shown.

Further, while the above cited approach delay and the related level of service (LOS), some jurisdictions will emphasize changes in the (v/c) ratio in their criteria.

If the instructor does not have simulation tools available, a good part of this analysis can be done by looking at a critical movement analysis and the sum of critical lane volumes, at the driveway and each existing intersection. It is likely that any need for additional lanes or phases will be revealed by such an analysis.

<u>Problem 14-3</u>

The driveway will have 400 vph in and 400 vph out. It is a busy activity, and internal circulation can be a major problem. Think in terms of one vehicle arriving every 9 seconds, but then allow that all those entering from the north will be platooned and those from the south will be more spread out --- if there is a NB right turn lane. But still, things can be very hectic immediately inside the site, particularly for any pedestrian traffic.

Now we come to a difficult but very realistic scenario --- the "sketch planning" concept of one driveway may not hold up under the reality of the traffic loads, once we look closely. Indeed, this may already have happened in addressing Problem 14-2, because the SB left turn lane into the driveway probably introduced significant major problems in signal timing, and pushed the limit for a single turning lane.

If so, it may be getting to be time to consider the problem statement as well-intentioned, but naïve. Two immediate thoughts come to mind:

1) Split the driveway into two, such as in Figure 14.2.b. Perhaps that can suffice, for both mainline and internal circulation issues;
2) Consider having a driveway on Avenue B, EB of Main Street. Oops, no – Avenue B is westbound, so the challenging SB left turn traffic can't use it. So, consider a Driveway on Avenue C, and adding protected turns to the Main Street SB at Avenue C.

Clearly, we are now talking about some lively discussion with the class. **The instructor should forewarn the class to never take a design problem statement at face value:** they must be ready to think of alternatives and to support them with sound arguments, especially if someone's "pet concept" is involved.

As to the internal circulation, much depends upon whether an approach like Figure 14.2.b is used, or a side-street driveway, or other. But the dissipation or routing of the arriving and departing flows in order to have reasonable parking-to-facility pedestrian traffic is quite important.

Another aspect is the required storage at the driveway exit (or exits, at more than one location). To maximize parking within the site, as well as the footprint of the actual building(s), shorter storage areas for queueing are often sought. But 200 vph exiting NB and 200 vph exiting SB can be a challenge, viewed on a per-cycle basis.

Problem 14-4

If the free flow speed were to be 60 fps, then $L/v = C/2$ would yield $C/2 = 1800/60 = 30$, or $C=60$ seconds for a very nice alternate progression, with the bandwidth fully used in both directions.

Some of the Table 14.3 volumes are in the range of 1100-1200 vph in the AM and PM, thru + right turn traffic, over 2 lanes. Allowing for the PHF = 0.85, it is unlikely that C=60 seconds will work very well when a critical movement analysis is done. But nonetheless, it would have been very nice, from the view of progressing traffic in both directions. Of course, Table 14.3 does show that the NB/SB flow is not equally balanced, so equal progression quality is not a necessity.

Cycle lengths of 80 or 90 seconds might fit well with this case, and with the overall network. A cycle length of 120 seconds might require needless waiting. Even if local practice is protected left turns, the volumes shown in Table 14.3 will not lead to very long turn phases.

The actual submitted work should look at all intersections, using critical movement analysis, HCM, or Synchro.

Problem 14-5

This is a not uncommon issue for the traffic engineer to address --- a favored design that has real operational problems. Despite the symmetry, the need for two heavily used 5-leg intersections will present problems with required cycle lengths, queueing and space for queueing, and delays. Thru traffic on the arterial will be caught up in the needed servicing of the site(s) in this favored design.

It is also likely to induce operational problems within the site.

Alert for the instructor: the authors did not introduce limits on geometry or available land that would preclude consideration of roundabouts at the 5-leg locations or even along the arterial, particularly as mitigation needs appear. Indeed, some jurisdictions may require explicit consideration of roundabouts and require justification for not using them. For this set of notes and discussions, the authors have assumed that local conditions (primarily space) will preclude roundabouts. But the topic should be mentioned.

Returning to the favored design, if it indeed creates more severe impacts than other designs, there will be an education process to move the developer and/or developer's architect away from the concept. In general, designs that respect certain preferences can find easier acceptance:

- **Thru traffic** that will bypass the site in any case should have be minimally impacted, if at all;
- **Access management** by use of side street entrances and such would preserve the original arterial function;
- **Roundabouts** can be an asset in some design concepts, although not presently anticipated by the authors;
- Local regulations in this particular problem statement greatly favor **future transit needs** and use, so embracing that as a reality is simply …… realistic.

The local requirements specify design for a much higher level of transit use than is currently common at such sites. The authors simply accept this as a given, and do not suggest trying to contest it: it may well be part of a long-term plan that will only be realized over an extended period, and the pieces must be put in place, step by step, to enable the future to be realized.

At least one of the authors can remember back to the early days of building accessible features into the infrastructure, apparently piecemeal. Consider:

- Curb cuts required, before any other feature;
- Door widths in buildings specified in new construction and in upgrades;
- Accessible restrooms;
- Ramps to access buildings;

- Buses that can accommodate wheelchairs;
- Elevators in transit stations;
- All other aspects of a total trip addressed, over time.

At one time, only fragments existed, from this list --- and it took decades. But without the effort, the barriers to the total trip would still exist. Indeed, still *do* exist in some ways. But today's undergraduates grew up in a world in which every one of the features listed above are a routine part of life, and of design.

Further, the reminder back then that "there is a spectrum of disabilities and infirmities, experienced by a wide range of people" is without question valid. Curb cuts are not just for wheelchairs but are used by people with strollers, shopping goods, walkers, and increasing caution with reliably stepping up the required distance at a curb.

We did an apparent digression, but not really: the argument that a need is not immediate, and is merely an aspiration, has been around for a very long time. Planning for major long-term shifts is valid, despite some inconveniences introduced today. The authors put what they view as an aggressive transit goal into Case Study 2 in order to focus the students on this concept of long-term planning and long-term benefit, by building a system over time.

Problem 14-6

Element 3 of Case Study 2 is somewhat mechanical, given the proper tools. But it has five discussion points listed for the student's attention. Every one of them needs attention in class.

This is an excellent point at which to remind the students that the process involves early meetings at which the **scope of the project is agreed upon with the reviewing agency and a formal written agreement is reached on that scope, issued by the appropriate authority.** It can be disastrous for everyone to proceed with an "implicit" understanding of the spatial extent of the project area is ---- how far away does it extend, which intersections and routes are involved, and so forth. The same is true of temporal extent --- traditional peak hours and mid-day, peak hours imposed by the construction or new site, future year(s) for analysis. In some cases, it is both a future build year and a peak construction year. None of this can be taken for granted, or left to implicit and/or oral agreements: reviewing officials change, new rules and regulations come into existence, one agency or even unit within an agency claims jurisdiction of being the "lead agency" for review (even after a long period of working with a different unit), central office points to limits on district offices that were never mentioned, and so forth. Even the client sometimes wants to move ahead without the "formality" because they seem to be making headway and can "iron out the details later". The traffic professional has to guide, sometimes bluntly, and sometimes by documenting the error of not having a formally accepted scope --- spatially, and temporally.

Problem 14-7

Element 4 of Case Study 2 is also mechanical on some level, but does include four discussion points for the student's attention.

References [6] and [7] were specified for trip generation rates, but the instructor may have to specify rates or direct the students to another site, if these references are not available locally.

It is expected that parking layout will be a challenge. As a result, considering the site as two distinct parcels with two different uses --- and thus buffer zones between the parcels, and possibly separate access points --- may be infeasible.

In dealing with parking and access, the students will also have to remember truck deliveries, waste removal, and such.

The point was made that the construction phase may require detailed attention in some cases, even beyond the future build case. One of the authors was involved in a few such projects, in which the construction phase was 4 to 5 years, and the construction workforce peaked at about 5,000 workers. The actual facilities (that is, the future build scenario) had a permanent workforce of less than 500, with biannual refueling of the power plant that added additional staff for a short period.

Problem 14-8

Element 5 of Case Study 2 overlaps with both Elements 4 and 6, because trips generated comes from the Section 14.5.2 specification that "The multiplex is to have eight theaters, of which four will have 400 seats and four will have 200 seats. The shopping mall is to be built out to the limits of the local code".

The last sentence above means that parking can be a limiting factor, or even site access capacity can be a limiting factor: people can't use the mall, if they can't get to it. Complicating matters is that space has to be reserved as if there were 20% transit access, and parking space has to be allocated for today's 5% transit.

The instructor should remind the students that the specification for the parking related to the mall is based upon gross floor area (GFA), **not** gross leasable area (GLA) and **not** gross usable area or any other term. GFA generally is measured as all of the square footage inside the building envelope to the outside of the walls but not including the roof. It includes "common areas" such as walkways, atriums, and other interior spaces. But

- In some jurisdictions, local codes define GFA as excluding areas for waste storage, enclosed maintenance vehicle parking, other interior parking, and loading dock work areas. But absent such a clear local code, the default is "all" as just cited.
- Some realtors use the terms GLA and GFA interchangeably, but leases are (hopefully) more precise.

To an objection that "But why assign parking spaces to the 'ambience' common areas of a mall?", the common answer is that if the code were rewritten in terms of GLA, it would simply have a larger number of parking spaces required per 1,000 square feet. Similar responses are often made to the distinction between "anchor" tenants that attract trips and secondary tenants that draw mainly from the anchor tenant traffic. This would then lead to a chicken-and-egg discussion (e.g. which came first?), but the bottom line is that the local code prevails as written.

The students may also need to be reminded that the peak loads of a multiplex and of a shopping mall do not necessarily occur at the same time, or even on the same days. Given local code requirements (parking based upon number of seats for the multiplex, GFA for the mall; see Table 14.4), it may take some discussion (hypothetically, with the local review agency; in this class setting, with the instructor playing that role) to argue that some parking spaces are dual-use, given the loading patterns.

The authors have seen some student solutions that included roof-top parking.

Yes, there is a bit of "simultaneous equation" solving involved: more GFA space means more parking space needed, means hitting capacity of the parcel(s) and/or less GFA, and so forth.

It also means more space for internal circulation, for access roads, for storage of exiting vehicles, and so forth.

And the need for attractive, integrated transit space at 20% as specified in the local code cannot be forgotten.

Which does remind one of the discussions that can be had about (vehicle) trips vis-à-vis persons visiting, and thus to vehicle occupancy --- assumptions, literature, and data-based estimates.

<u>Problem 14-9</u>

Element 6 relates to driveway locations, special arterial design features, and special intersection design features. It flows from, and is intertwined with, Element 5.

The student should be encouraged to think of preferred paths for goods vehicles, transit, and personal cars. Are they distinct, and to what extent? How can transit service be thought of as part of overall local routes (now, and of the future)? Should proximity to the facilities be given to transit (the reviewer's expectation will likely be "yes") but how does this affect walking times from personal vehicles, and safe pedestrian paths?

In a large project of this sort, in real practice there may be other professional specialists involved, including architects, site planners, corporate marketing, and of course the client with their own experience base. In this exercise, the instructor may have to provide the other, distinct views that challenge the transportation solutions put forth.

Future editions of the textbook may be able to deal with self-parking semi-autonomous cars and/or need to. But not necessarily for this 5th edition. However, during the careers of today's students, such vehicles may be a reality: with this in mind, some radical rethinking of parking, parking space, and distances is already being done ----- self-parking cars do not need to open their doors at the storage end of the internal trip; self-parking cars do need to un-park with sufficient notice to arrive at the meeting point with the owners; and so forth. While it is not necessary to address these aspects in <u>this</u> assignment, it is probably appropriate for the course to contain some "think of a different future, with different opportunities and solutions" moments.

<u>Problem 14-10</u>

Element 7 is mitigation. The text lists three discussion points. For present purposes, we now emphasize two related thoughts:

1) The authors have seen some site developers focus too much on "how do we get the approvals, and move ahead" and less on "Will this plan help make my site an attractive destination for the future customers, and thereby a success?" The traffic professional may sometimes have to remind the client of this other perspective. This can be a challenge when the client's management structure appoints a client team charged with great emphasis to "Get it done, by the deadline."
2) Mitigation can be the driver for good design.

Of course, reality is also that the developer likely has to pay for the improvements, including those that go beyond the site boundaries. A continual focus on the differences between "future build" and "future no build" has to be re-iterated ---- despite the direction that some meetings will take, comparing "existing" and "future build" conditions places an unreasonable burden on what is expected of the developer.

<u>Problem 14-11</u>

The authors have found that two presentations are sometimes of most value to the students.

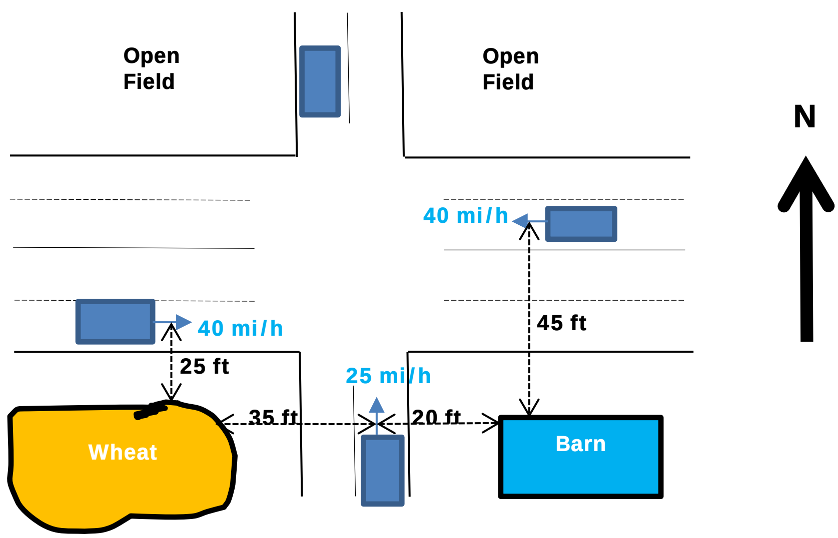
The first is with the instructor only, playing the role of a mildly disbelieving reviewer or even a cantankerous one. It is an "in the family" review. Assumptions are challenged, and the student needs to be ready to cite standard references and their use. Students tend to use "I" too much (I did this, I did that); they have to be moved away from that to "we" and "the team". There will be nervousness, and lame humor. Eye contact, volume and speed of speaking, and such are likely to be issues. The authors have found it useful to remind students that this first draft presentation is to better prepare for the real show, with outsiders. The first presentation, even if challenging, is "within the family".

The second presentation would best involve some outside professionals, perhaps from an agency or from a consultant experienced in this work. But those visitors have to remember that they are helping beginners, by their role-playing. Expectations cannot be for flawless results. At the same time, the ability to respond to the outsiders is part of the evaluation, and the grade. We have limited the visitors to one or two; after that, it gets lengthy and even disruptive.

Solutions to Problems in Chapter 15

The Hierarchy of Intersection Control

Problem 15-1



The solution begins by assuming that the minor street vehicle, the NB vehicle in this case, is located one safe stopping distance from the collision point:

$$d_{NB} = 1.47\,S_{NB}\,t + \frac{S_{NB}^2}{30(0.348 + 0.01G)}$$

$$d_{NB} = (1.47 * 25 * 2.5) + \left[\frac{30^2}{30*(0.348 + 0.01*0)}\right]$$

$$d_{NB} = 91.88 + \left(\frac{900}{10.44}\right) = 91.88 + 86.21 = 178.09 \ \ ft$$

There are two sight triangles that must be checked: (a) the sight triangle between the NB and WB vehicle, and (b) the sight triangle between the NB and EB vehicle. Sight triangles involving the SB vehicle have no obstructions, and may be assumed to be acceptable.

(a) Sight Triangle Between NB and WB Vehicles

Using the geometry of the sight triangle, the distance of the WB vehicle from the collision point when the two drivers first see each other may be established:

$$d_{WB}\,(act) = \frac{a\,d_{NB}}{d_{NB} - b} = \frac{20*178.09}{178.09 - 45} = \frac{3{,}561.8}{133.09} = 26.76 \ \ ft$$

What this means is that when a NB vehicle is 178.09 ft away from the collision point, the drivers of the NB and WB vehicles can first see each other when the WB vehicle is 26.76 ft away from the collision point.

There are two potential safety rules. Rule 1 requires that the WB vehicle also be one safe stopping distance from the collision point when the two drivers can first see each other:

$$d_{WB}(\min, Rule1) = (1.47*40*2.5) + \left(\frac{40^2}{30*0.348}\right)$$

$$d_{WB}\ (\min, Rule1) = 147.00 + \left(\frac{1{,}600}{10.44}\right) = 147.00 + 153.26 = 300.26\ \ ft$$

The second safety rule requires that the vehicles be able to safely pass, one behind the other, over the collision point. It uses Equation 15-3 of the textbook:

$$d_{WB}(\min, Rule2) = (d_A + 18)\frac{S_B}{S_A} + 12$$

$$d_{WB}(\min, Rule2) = (178.09 + 18)\frac{40}{35} + 12 = 325.74\ ft$$

As both of the minima required by Rules 1 and 2 are considerable larger than the actual distance of the WB vehicle from the collision point, this situation must be deemed *unsafe*. Basic rules-of-the-road *may not* be applied.

Since *all* sight triangles must be safe in order to allow imposition of basic rules-of-the-road, no further computations are necessary. For illustrative purposes, the analysis of the sight triangle between the NB and EB vehicles will be done.

(b) *Sight Triangle Between the NB and EB Vehicles*

Once again, computations begin with the NB vehicle assumed to be one safe stopping distance from the collision point, or 178.09 ft. The actual distance of the EB vehicle from the collision point when the two drivers can first see each other is now established:

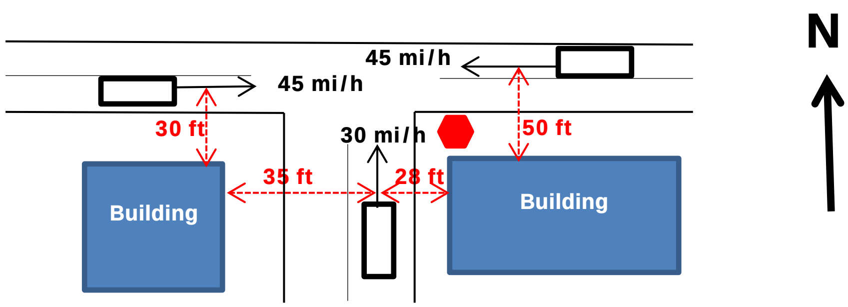
$$d_{EB}(act) = \frac{a\,d_{NB}}{d_{NB} - b} = \frac{35*178.09}{178.09 - 25} = \frac{6{,}233.15}{153.09} = 40.72\ ft$$

The minimum distance that the EB vehicle *should* be from the collision point is determined using either Rule 1 (safe stopping distance) or Rule 2. The Rule 1 minimum distance is the same as for the WB vehicle, as they have the same safe stopping distances: 300.26 ft. The Rule 2 distance is computed using Equation 15-3 of the textbook, and is also the same as for the WB vehicle: 325.74 ft.

Again, both minimum distances are much larger than the actual distance of 40.72 ft. Therefore, this sight triangle is also deemed to be unsafe, and basic rules-of-the-road may not be implemented.

It should be noted that there is little chance that vehicles that are approximately 178 ft from the collision point will actually hit vehicles that are approximately 40 ft from the collision point. What the analysis of the sight triangle shows, however, is that there could be vehicles on a collision path with each other when they cannot see each other. *That* is what is unsafe. Drivers cannot be expected to use judgment alone to avoid an unsafe situation if they cannot see it.

Problem 15-2



The analysis of sight distance when a STOP sign is in place differs from the analysis of the sight triangle for basic rules-of-the-road. Vehicles are stopped when they look for a gap in conflicting traffic through which to pass.

The driver in the STOP-controlled vehicle is assumed to be stopped at a location that is 18 ft from the intersection curb line. This consists of an assumed 10 ft between the STOP line and the curb line, and an assumed 8 ft for the distance between the driver's eye and the end of his/her vehicle.

The distance between the driver's eye and the conflicting driver's line of sight is computed as:

$$d_{A-STOP} = 18 + d_{cl}$$

where d_{cl} is the distance between the curb line and the centerline of the nearest conflicting vehicle lane. With 12-ft lanes on the E-W street, this would be 6 ft for the EB vehicle and 12+6 = 18 ft for the WB vehicle. Thus:

$$d_{A-STOP,EB} = 18 + 6 = 24 \ \ ft$$
$$d_{A-STOP,WB} = 18 + 18 = 36 \ \ ft$$

This becomes the assumed distance of the NB vehicle from the possible collision points.

The minimum distance for conflicting vehicles on the through street is based upon gap acceptance theory, according to the following equation:

$$d_{B\min} = 1.47 * S_{maj} * t_g$$

For typical situations, a passenger car turning left from the STOP-controlled approach will require a gap of 7.5 s. Therefore:

$$d_{B\min} = 1.47 * 45 * 7.5 = 165.38 \ ft$$

This must be compared to the actual distance of EB and WB vehicles from the collision point(s) when sight lines are first established. As in the case of a basic rules-of-the-road analysis, the geometry of the sight triangles is used to determine this:

$$d_{EB}(act) = \frac{a\, d_{NB}}{d_{NB} - b} = \frac{35 * 24}{24 - 30} = \frac{840}{-6} = -140 \ ft$$

$$d_{WB}(act) = \frac{28 * 36}{36 - 50} = \frac{1{,}008}{-4} = -252 \ ft$$

The negative answers in both cases indicate that sight lines are actually unobstructed. The installation is safe, and no changes need to be made.

<u>Problem 15-3</u>

In this intersection, there are only two sight triangles, both of which are obstructed. The one that appears to be most restricted is to the left of the vehicle approaching on the one-way street. It will be analyzed first.

The vehicle on the one-way street is Vehicle A. It will be placed one safe-stopping distance from the collision point:

$$d_A = (1.47 * 30 * 2.5) + \frac{30^2}{30(0.348 + 0.045)} = 110.3 + 76.3 = 186.6 \ ft$$

From the sight triangle, the actual distance of Vehicle B from the collision point when both drivers can see each other is:

$$d_b(act) = \frac{20 * 186.6}{186.6 - 20} = 22.4 \ ft$$

This must now be compared to the two minimum conditions:

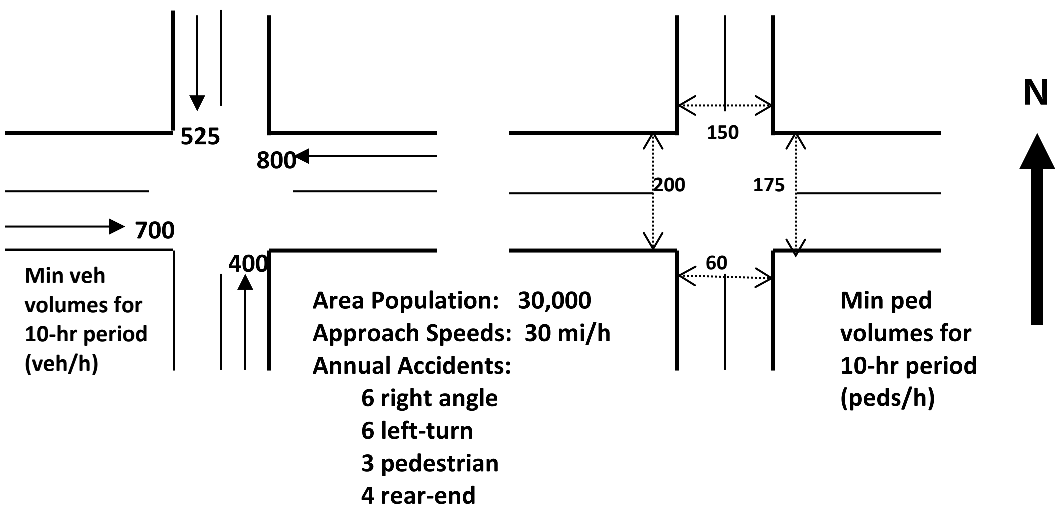
$$d_{b\min}(Rule\ 1) = (1.47*45*2.5) + \frac{45^2}{30*0.348} = 165.4 + 194.0 = 359.4\ \ ft$$

$$d_{b\min}(Rule\ 2) = (186.6+18)\left(\frac{45}{30}\right) + 12 = 318.9\ \ ft$$

Neither rule for safe operation is met. Therefore, operation under basic rules of the road is not safe. In most cases, a STOP-sign would be recommended.

As in Problem 15-1, the second sight triangle could be analyzed, but it is not necessary, as the intersection has already failed the safety test for one sight triangle.

Problem 15-4



The data for this problem are in an interesting form: minimum volumes that apply over a 10-hour period. Because of this, the single volume point provided covers 10 hours. However, because they are *minimum* volumes, some of the hours may (in fact, probably do) have *higher* volumes. Thus, if the volumes given meet a warrant criteria, that will hold for 10 hours. If, however, the volumes given *do not* meet a specific warrant, we cannot say definitively that the hour(s) in question do not meet the warrant. They may, but we have insufficient data to make the determination.

Note that neither the area population nor approach speeds qualify the intersection for consideration of warrants at the 70% level. All analyses for this intersection will refer to the 100% criteria given for each warrant.

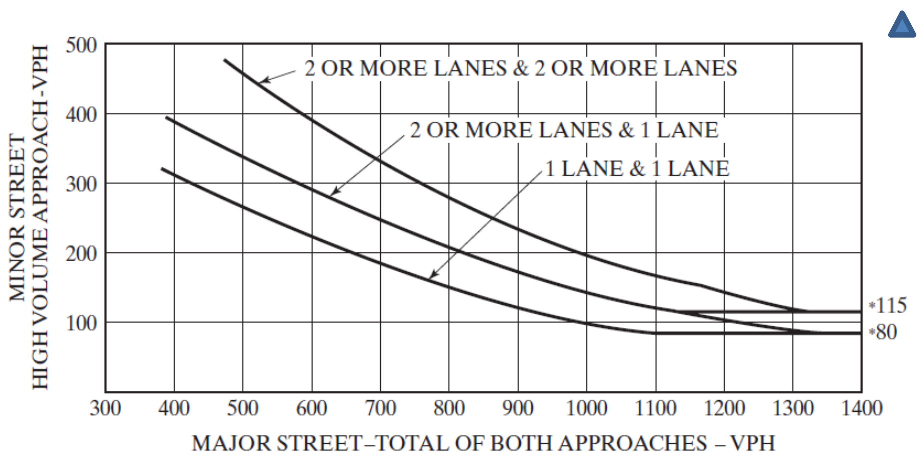
Warrant 1: Eight-Hour Volumes

The hourly criteria use the total 2-way volume on the main street vs. the high directional volume (1 direction) on the minor street. Because of the higher volumes, the E-W street will be treated as the major street in this analysis. The 100% volume criteria for this warrant has two volumes: (1) Condition A – Crossing Volumes, and (2) Condition B – Interruption of Continuous Traffic. The 100% criteria are:

	(Main street 2-way, Minor street 1-way)
Condition A:	(500, 150)
Condition B:	(750, 75)

The actual volumes are (700+800,525) or (1500,600). Both Conditions A and B are satisfied for at least 10 hours, and the warrant may be deemed to be **MET**.

<u>Warrant 2: Four-Hour Volumes</u>
The actual volume (1500,525) is plotted on Figure 15.5(a) as shown below, and compared to the 1 lane x 1 lane decision line:



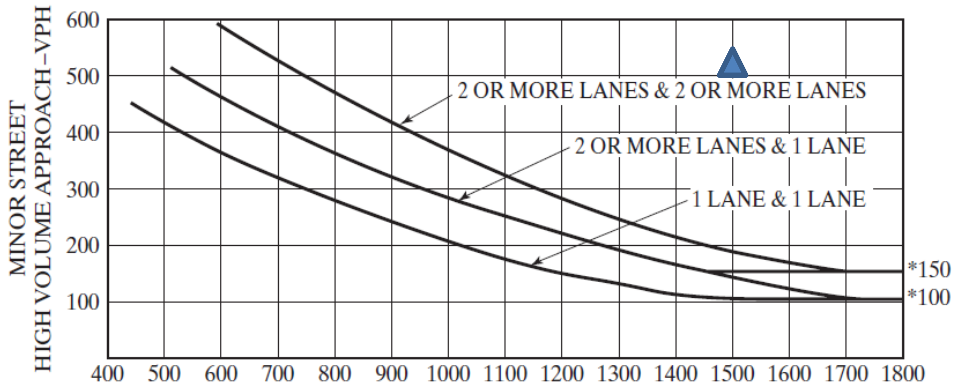
*Note: 115 vph applies as the lower threshold volume for a minor street approach with two or more lanes and 80 vph applies as the lower threshold volume for a minor street approach with one lane.

(a) Normal Conditions

The point, which represents 10 hours of data, is off the chart, and obviously above the decision line. The warrant may be deemed to be **MET**.

<u>Warrant 3: Peak Hour</u>
This warrant has two parts. The delay criteria do not apply for two reasons: No delay data is given, and there is no STOP sign already in place.

The peak hour volume criteria is checked by plotting the volume point on Figure 15-6(a) and comparing it to the 1 lane x 1 lane decision line.



MAJOR STREET – TOTAL OF BOTH APPROACHES – VPH

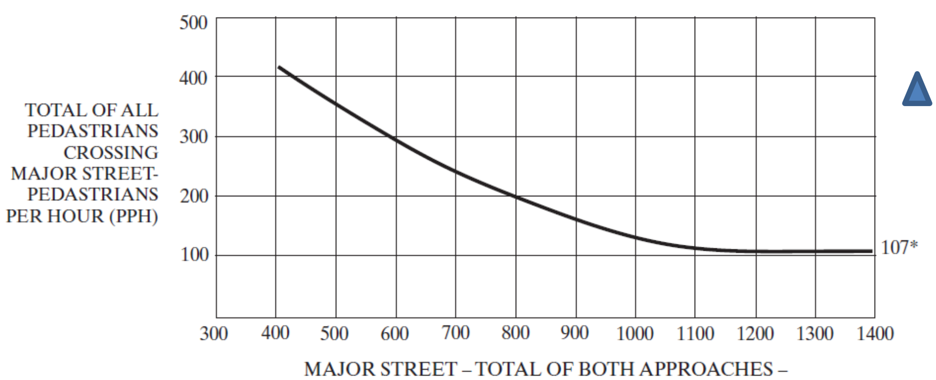
*Note: 150 vph applies as the lower threshold volume for a minor street approach with two or more lanes and 100 vph applies as the lower threshold volume for a minor street approach with one lane.

(a) Normal Conditions

The point is obviously over the decision line, so the warrant may be deemed to be **MET**.

Warrant 4: Pedestrian Volumes

The volume point consisting of (total veh. volume on major street, peds xing major street.) must be plotted on Figures 15-7(a) – the four-hour pedestrian criteria, and 15-8(a) – the one-hour pedestrian criteria and compared to the respective decision lines on these curves. The point is (1500,200+175) or (1500,375), which represents minima for 10 hours of the day.

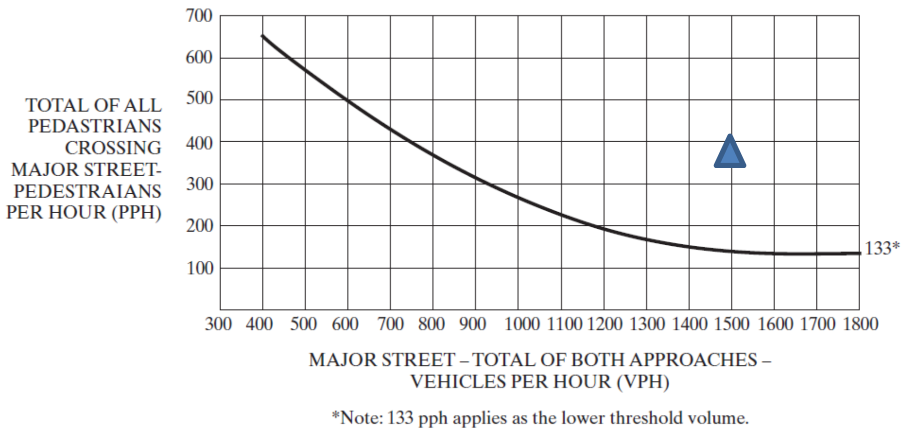


MAJOR STREET – TOTAL OF BOTH APPROACHES – VEHICLES PER HOUR (VPH)

*Note: 107 pph applies as the lower threshold volume.

(a) Normal Criteria.

4-Hour Criteria



One-Hour Criteria

Obviously, both the 4-hour and 1-hour criteria are met. This warrant may be deemed to be **MET**.

<u>Warrant 5: School Crossing</u>
This warrant does not apply, as this is not a school crossing.

<u>Warrant 6: Coordinated Signal System</u>
No system information is given, so this warrant cannot be assessed.

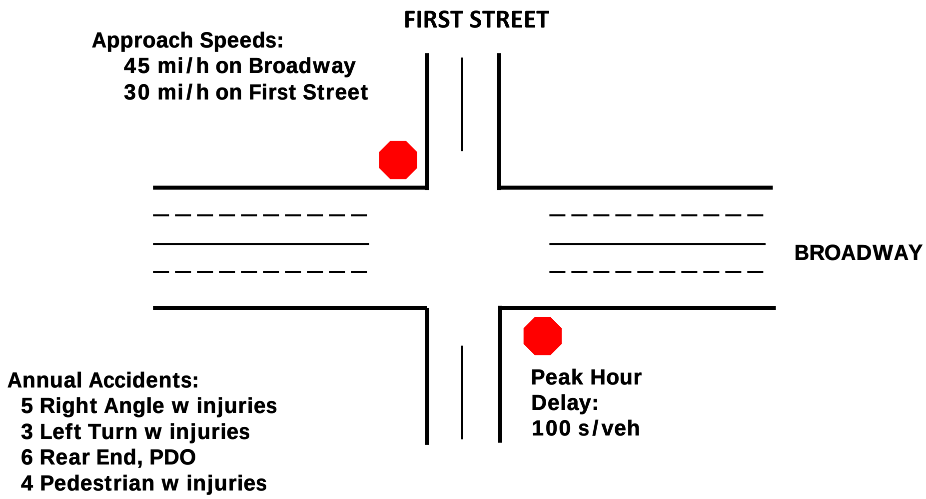
<u>Warrant 7: Crash Experience</u>
While the accident data would normally meet this warrant (there are 15 total accidents susceptible to correction through signalization), and the volume requirements are met at 100%, there has been no trial of alternative measures (STOP, YIELD control). In the absence of such a trial, this warrant must be deemed to be **NOT MET**.

<u>Warrant 8: Roadway Network</u>
Warrant 8 deals with projected volumes. None are given. This warrant does not apply.

<u>Warrant 9: Railroad Crossing</u>
This is not a railroad crossing location. This warrant does not apply.

<u>Recommendation</u>
A signal should clearly be installed, as several warrants are met with much room to spare. Because the geometry has only one lane on each approach, a two-phase signal would be implemented. It could be actuated or pretimed, based upon system information not given here. Because the pedestrian warrant was triggered, use of pedestrian signals is clearly indicated.

Problem 15-5



Volume Data

Time	Volume on Broadway (veh/h)			Volume on First Street (veh/h)			Peds Xing Broadway (Peds/h)
	EB	WB	Total	NB	SB	High Vol	
10-11 AM	730	700	**1430**	300	400	**400**	**140**
11-12 AM	775	700	**1475**	300	400	**400**	**150**
12-01 PM	800	710	**1510**	315	410	**410**	**190**
01-02 PM	800	715	**1515**	325	420	**420**	**210**
02-03 PM	820	720	**1540**	350	450	**450**	**220**
03-04 PM	830	725	**1555**	360	450	**450**	**220**
04-05 PM	900	780	**1680**	400	480	**480**	**200**
05-06 PM	925	790	**1715**	410	520	**520**	**200**
06-07 PM	950	800	**1750**	375	510	**510**	**230**
07-08 PM	950	800	**1750**	350	480	**480**	**250**
08-09 PM	940	750	**1690**	320	420	**420**	**220**
09-10 PM	880	700	**1580**	306	400	**400**	**190**
10-11 PM	750	690	**1440**	295	390	**390**	**140**
11-12 PM	650	630	**1280**	260	380	**380**	**100**

It is obvious in this case that Broadway is the major street, and First Street is the minor street. The data is already organized with that in mind, and the necessary subtotals are already provided.

Because of the 45-mi/h speeds on Broadway, the 70% criteria for all warrants (where they are available) should be used.

Warrant 1: Eight-Hour Volumes

Using the 70% criteria, Warrant 1 had two requirements (Condition A, Condition B). Meeting one of these is sufficient to meet warrant. The criteria, in the form of (major street 2-way volume, minor street 1-way high volume) for a 2 x 1 intersection are:

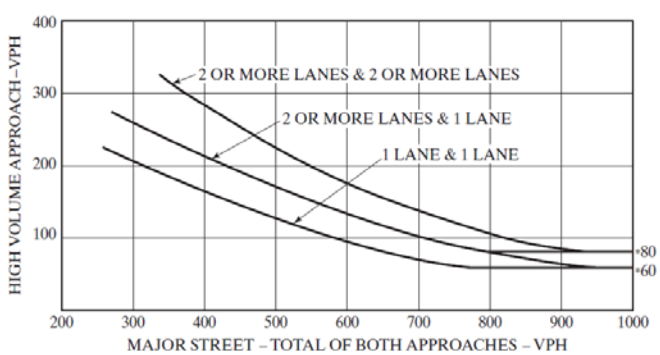
Condition A:	(420,105)
Condition B:	(630,53)

In the volume table provided, *all 14 hours* meet the major street volume criteria for both conditions. *All 14 hours* also meet the minor street volume criteria for both conditions. Both conditions are met, and the warrant may be deemed to be **MET**.

Warrant 2: Four-Hour Volumes

Each hourly data must be plotted on Figure 15-5(b) and compared to the decision line for a 2 lane x 1 lane intersection. Four points must lie above the decision line for the warrant to be met. Rather than plotting all 14 hours, the worst 4 hours are plotted. If the worst 4 do not all lie above the decision line, then none of the others will either.

Unfortunately, the worst 4 hours are not clear, because the major and minor street traffic do not peak at the same time. It is clear, however, that the worst 4 hours occur between 4 PM and 9 PM, a period covering five hours. These will be plotted.



*Note: 80 vph applies as the lower threshold volume for a minor street approach with two or more lanes and 60 vph applies as the lower threshold volume for a minor street approach with one lane.

(b) Criteria for Small Communities (pop <10,000) or High Major Street Approach Speed (≥40 mi/h)

Obviously, all five points (not shown due to the scale) fall well off the curve and above the 2 x 1 decision line. The warrant may be deemed to be **MET**.

Warrant 3: Peak Hour

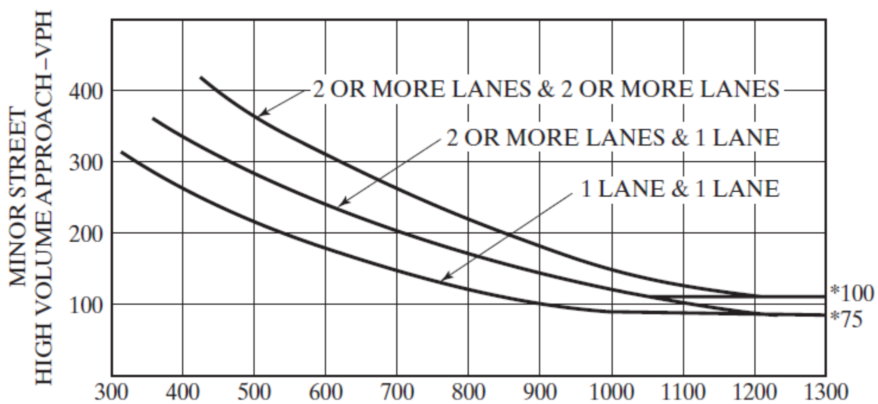
The peak hour warrant has two parts, both of which may be evaluated in this case.

Peak Hour Delay: During the highest minor street volume hour, there are 520 veh/h in one direction on a STOP-controlled approach, each of which is delayed by 100 s. The total delay experienced by these vehicles is, therefore:

$$delay = 520 * 100 = 52{,}000\ veh - s = \frac{52{,}000}{3{,}600} = 14.4\ veh - hrs$$

This is in excess of the criteria of 4 veh-h of delay to trigger the warrant. The volume requirements for use of the delay criteria are also met. The delay criteria are, therefore, met.

Peak Hour Volume: The worst hour of the day is either 5-6 PM or 6-7 PM. These volumes are plotted on Figure 15-6(b) and compared to the 2 x 1 decision line.



*Note: 100 vph applies as the lower threshold volume for a minor street approach with two or more lanes and 75 vph applies as the lower threshold volume for a minor street approach with one lane.

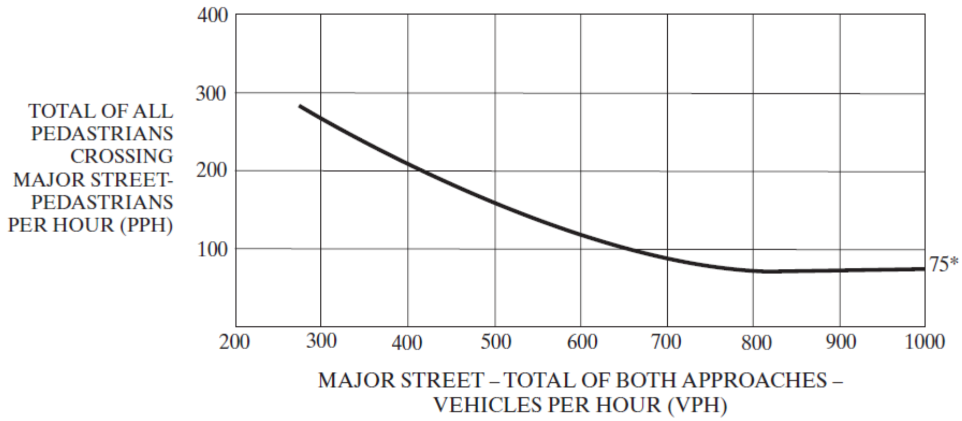
(b) Criteria for Small Communities (Pop <10,000) or High Major Street Approach Speed (≥40 mi/h)

Again, both points, if plotted, would be well off the chart and above the 2 x 1 decision line. The volume criteria are met.

Because both criteria are met, this warrant may be deemed to be **MET**.

Warrant 4: Pedestrian Volumes

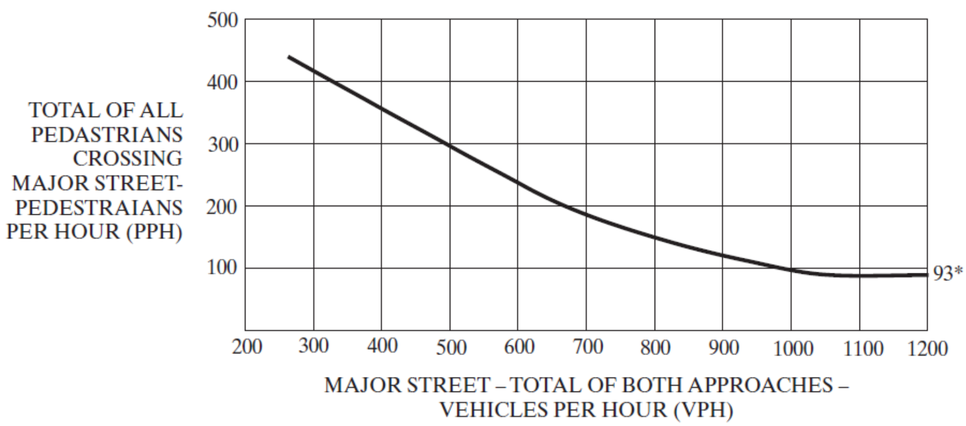
Volume points are compared to the criteria in Figures 15-7(b) – four hours, and 15-8(b) – one hour. The points are plotted as (total major street vol, peds xing major street). Attention will be focused on the worst four hours, which appear to be between 5 PM and 9 PM (in terms of the vehicular-pedestrian conflict).



*Note: 75 pph applies as the lower threshold volume.

(b) Criteria for Small Communities (Pop <10,000) or High Major Street Approach Speed (> 35 mi/h)

4-Hour Criteria



(b) Criteria for Small Communities (Pop <10,000) or High Major Street Approach Speed (> 35 mi/h)

1-Hour Criteria

Once again, the points, if plotted, would lie well off of the charts, and clearly over the decision lines. Both criteria are met. This warrant may be deemed to be **MET**.

Warrant 5: School Crossing
This location is not a school crossing. The warrant does not apply.

Warrant 6: Coordinated Signal System
No signal system information is provided. This warrant may not be assessed.

Warrant 7: Crash Experience
There are 18 accidents per year at this location. Of these, only the 6 rear-end collisions would not be susceptible to correction through signalization. Further, there is a STOP-

sign already in place, and the volume requirements of this warrant are all met at 100%. This warrant may be deemed to be **MET**.

Warrant 8: Roadway Network
This warrant deals with forecast future volumes, none of which are provided. It does not apply.

Warrant 9: Railroad Crossing
This is not a railroad crossing. The warrant does not apply.

Recommendation:
Once again, several warrants are met with room to spare. A traffic signal should be placed at this intersection. As the pedestrian warrants are triggered, and there are pedestrian accidents, pedestrian signals should be used. Depending upon system considerations not given here, the signal could be pretimed or actuated.

Problem 15-6
Once again, only vehicular volumes are given. Non-volume-based warrants cannot be evaluated. Only Warrants 1 – 3 may be evaluated with the information given. Note that neither the population nor the approach speeds engage a reduction in criteria, so warrants must be met at 100% in this case. To assist in making this evaluation, the table should be re-arranged to show total 2-way volume on the major street (N-S) and the highest single-direction volume on the minor street (E-W). The table that follows shows this.

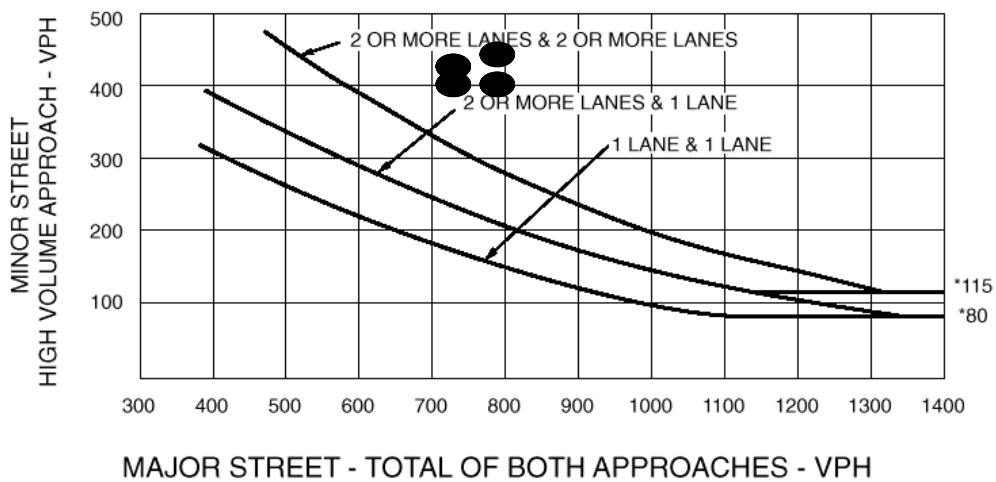
Table: Volumes for Warrant Analysis

Hour	Major Street Vol (2-Way)	Minor Street Vol (High Dir)
1	50	30
2	100	30
3	175	50
4	300	50
5	450	100
6	700	250
7	850	400
8	850	450
9	750	375
10	400	300
11	300	300
12	300	150
13	300	100
14	350	100
15	350	100
16	450	250
17	600	325
18	700	375
19	800	400
20	800	425
21	400	325
22	200	150
23	100	100
24	100	50

Warrant 1   Warrant 1, Condition A requires minimum volumes of 600 veh/h on the major street (2 ways) and 150 veh/h (one way) of the minor street. Condition B requires 900 veh/h and 75 veh/h respectively.

Hours 6, 7, 8, 9, 17, 18, 19, 20 meet Condition A (8 hours). No hours meet Condition B. The warrant is met.

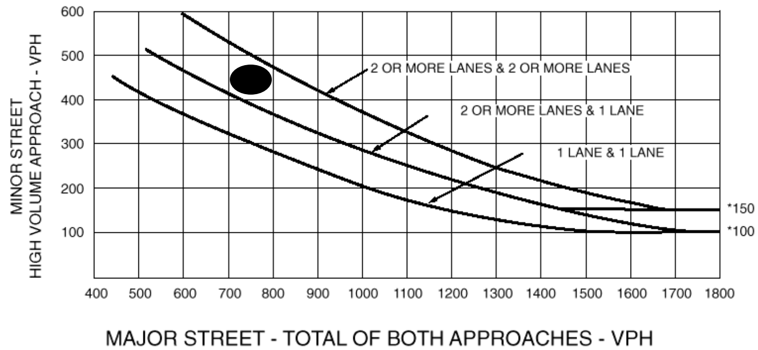
Warrant 2   While all 24 hourly points could be plotted against the 4-hour volume criteria, if the top 4 don't meet the warrant, no other set will meet the warrant. Hours 7, 8, 19 and 20 appear to be the worst periods. These four points are plotted on the figure below: [850,400], [850,450], [800,400], and [800, 425].



*Note: 115 vph applies as the lower threshold volume for a minor street approach with two or more lanes and 80 vph applies as the lower threshold volume for a minor street approach with one lane.

As all four points are clearly above the decision line, this warrant is met.

Warrant 3   Warrant 3 has two parts: peak hour delay, and peak hour volume. There is no delay information given, so the first part cannot be evaluated.   The second can be evaluated. The highest volume point [850,450] is plotted. If this hour does not meet the criteria, no other hourly volume pair will.



*Note: 150 vph applies as the lower threshold volume for a minor street approach with two or more lanes and 100 vph applies as the lower threshold volume for a minor street approach with one lane.

As the intersection has 2 lanes (each direction) on the major street and 1 lane (in each direction) on the minor street, the middle decision line is applicable. The warrant is met.

A signal is warranted by all three of the volume criteria. No particular form of signalization is recommended without additional information.

Problem 15-7

Because of the 45-mi/h speeds on the major street (E-W), the 70% criteria of the volume warrants apply. Because the minor street is a one-way street, the "total" volume is the "highest directional volume."

Warrant 1 Condition A requires minimum volumes of [420, 105]. Condition B requires [630, 53].

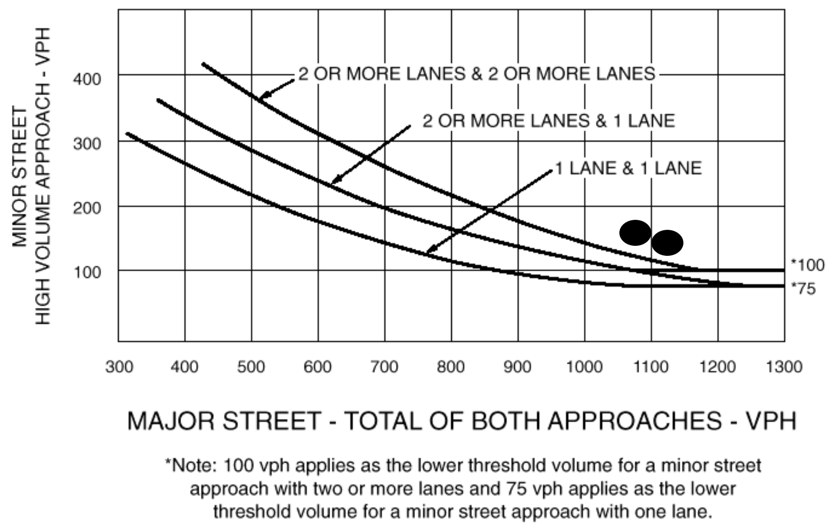
Condition A is met by the following hours: 3-4 PM, 4-5 PM, 5-6 PM, and 6-7 PM. This is only 4 hours, while 8 are required. Condition A is not met.

Condition B is met by the following hours: all hours between 1 PM and 11 PM. This is 10 hours. Condition B is met.

The warrant is met.

Warrant 2 The highest four-hour volume period is between 3 PM and 7 PM. These four hours are plotted against the 4-hour vehicular volume warrant criteria. As all of these points are off the volume scale on the 70% criteria for Warrant 2, and the minor street volumes are above the minimums required, the warrant is met.

Warrant 3 The potential highest volume hours [1150, 160] or [1200, 135] are plotted against the peak hour vehicular volume warrant criteria (70% level).

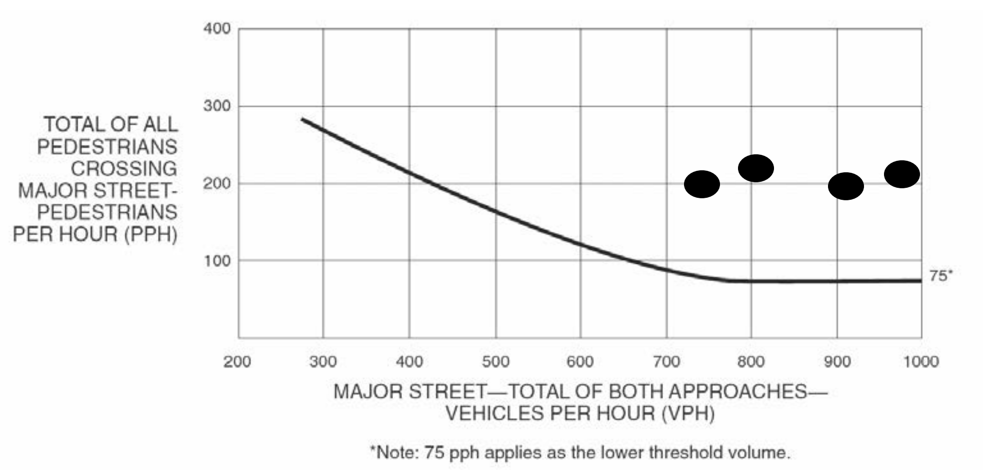


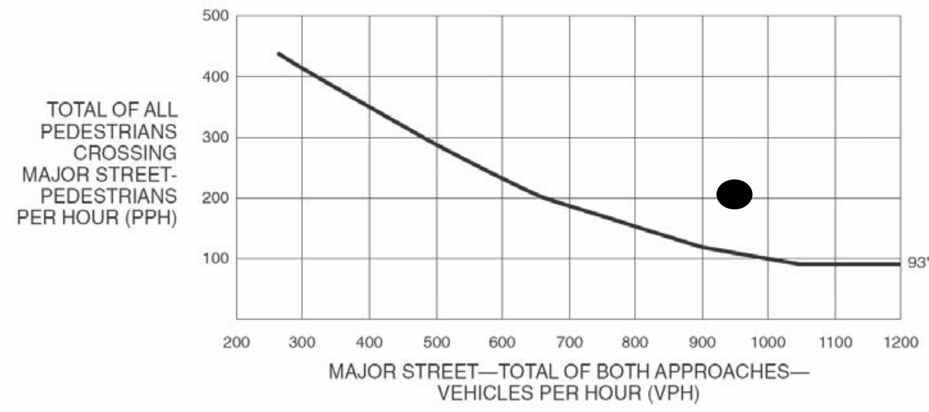
As both points lie above the decision line, the warrant is met.

The delay portion of Warrant 3 can also be evaluated for vehicles on the STOP-controlled approach. In the peak hour, 160 vehicle experience 72 s/veh of delay for a total of 72*160 = 11,520 veh-sec, or 3.2 hours of aggregate delay. The warrant requires a minimum of 4.0 hours, so this part of Warrant 3 is not met.

The warrant is met, as the volume criterion is met.

Warrant 4 The highest four hours of pedestrian activity occur between 1 and 4 PM, and between 8 and 9 PM. During these four hours, the major street vehicular volume and pedestrian volumes (crossing the major street) are [800, 200], [855, 210], [1025, 205] and [975,200]. These are plotted against the four-hour pedestrian warrant (70% level). The highest period [1025, 205] is plotted against the one-hour pedestrian warrant (70% level).





As both criteria are met, this warrant is met.

Warrant 5 The School Crossing Warrant is not applicable.

Warrant 6 The Coordinated Signal Warrant is not applicable.

Warrant 7 The Crash Experience Warrant can be evaluated. All relevant criteria are met: There is STOP-control in place, there are 8 right turn, 3 left turn, and 4 pedestrian accidents that can be corrected by signalization, and Warrant 1B is met at 100%. The warrant is met.

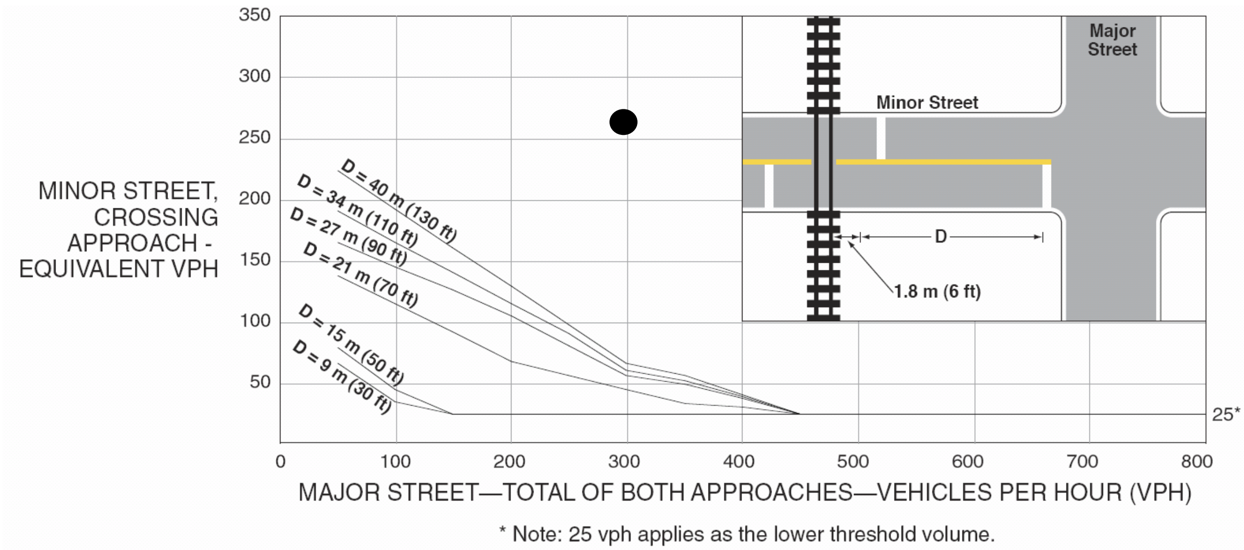
Warrant 8 The Roadway Network Warrant is not applicable.

Warrant 9 The RR Grade Crossing Warrant is not applicable.

A signal is clearly warranted at this location. Given the high pedestrian volumes, use of pedestrian signals is suggested for crossing the major street. If an actuated controller is used, a pedestrian actuator should be provided.

Problem 15-8

The first part of the solution is to determine the “equivalent” volume crossing the tracks. There are no buses, but there are tractor-trailers. There is also an adjustment for train frequency. For 20 trains per day, from Table 15-11, an adjustment of 1.33 is applied. For 20% tractor-trailers, from Table 15-13, an adjustment of 1.35 is applied. Thus, the equivalent volume crossing the tracks is 150*1.33*1.35 = 269 veh/h. This is plotted against 300 veh/h on the major street on the Figure 15-9 criteria curve:



The warrant is clearly met, and a signal should be placed. It should be coordinated with the RR crossing signals and gates.

Solutions to Problems in Chapter 16

Traffic Signal Hardware

Problem 16-1

This question is straightforward, but for many students (and others) it is confusing as to why there is not a single, national MUTCD. It seems so logical.

But the relationship between the individual states and the Federal Government is codified in the 10^{th} Amendment of the US Constitution:

> "The powers not delegated to the United States by the Constitution, nor prohibited by it to the states, are reserved to the states respectively, or to the people".

This applies to things big and small. And over time, there have been lively discussions and debate over the limits of Federal power. But the underlying principle is as quoted above. The US is a federation of the individual states.

So, there are "model ordinances" written by national committees, for guidance of individual local governments. And there is a national committee that creates the *Highway Capacity Manual,* but it is the jurisdiction of the state governments to decide to adopt it (or not). In many states, state law assigns this decision to the state transportation commissioner, so that new legislation is not required every time key specialty references are updated. And so it is true with the MUTCD, also. As one example, in NYS the state law authorizes the State DOT commissioner --- except that cities with populations of over one million can make their own decision. So, NYS can adopt the national model with some modifications but NYC can decide to follow the national model exactly.

It is true that the Federal Government has sometimes tied its funding to the state having a rule or adoption that "is in substantial compliance with" the national model. Given that none of the authors are attorneys, we will leave the discussion of this requirement vis-à-vis the 10^{th} Amendment to others.

So, the student's responsibility is to seek information on whether a specific state has simply adopted the national model of an MUTCD, or modified it, and in what way. The easiest way to address the problem is to go the State DOT web site, and search for information on the MUTCD. At one time, NYS (as an example) had a totally different printing of its MUTCD (8 ½ x 11 paper, and some differences) but moved to a shorter "Supplement" to the national model.

Problem 16-2

The problem statement clearly says "there is divided opinion in the profession on this assertion". Over the time between the 5th and 6th editions of this textbook, there will probably be another edition of Reference [1], and there will be additional reports on the subject of this problem.

To start the student on the way, some words on the two views are relevant:

> On one hand, it is true that signal plans are generally not updated as often as changing traffic demand and/or path selections might dictate; this is usually related to budget availability and prioritization. So, if the traffic signal system is designed to be adaptive or even "self-learning", wouldn't it follow that such responsiveness will systematically push the system to always seek certain adaptations on a regular, persistent basis.
>
>> One can acknowledge that this is logical but at the same time acknowledge that the traffic control system has to be driven to these settings each and every day. That is, the system must detect the need and respond to it, incurring delay during that time.
>>
>> But still, the argument would go, that is better than a static plan, isn't it? That leads to the question of how many truly static plans still exist. As of this writing, the answer is "many". And it is also true as of this writing that many adaptive systems are built upon a default time of day plan, or even use a library of alternative plans --- so the adaptation is choosing amongst them, and allowing some further adaptation on top of that selection.
>>
>> Still, there are limits to the range of adaptation usually allowed, whether it is by g/C ratios or cycle length or phasing.
>>
>> And there is a school of thought that would seek to have the adaptive system to be "self-reporting": if it is pushed to certain limits regularly (to be defined) or to certain patterns (again, to be defined), then it could trigger an alert that would result in the de facto settings becoming a new base plan, or in a re-evaluation of the default starting point.
>
> On the other hand, some would argue that a truly adaptive system would not encounter any of these problems and might not be tied to any progressive or other underlying plan – including the use of a common cycle length.
>
>> At the time of this writing, there are some highly adaptive designs that proponents would say address this well. And there are those who are concerned such systems do not make enough use of underlying patterns

and/or totally ignore them and/or optimize local performance at the expense of the system performance.

Those who actually operate traffic control systems are often very aware that the public and the media are very focused and very vocal. Systems and designs that are perceived as creating localized problems, or being insensitive to changing demand, cause headaches for the system operator. One very experienced sage in real operations took the view that the ultimate metric is the public --- if there is no clamor because of perceived chaos or excessive delay/stops, then the system is working.

While the above material provides background, it is the student who will have to sort out the successes and failures, and the progress (or lack thereof) toward self-learning systems that can cope with infrequent retiming. But the caution has to be that not all representations can be taken at face value.

Problem 16-3

Three dedicated left turns are allowed, and a web search on “triple left turn lanes” will yield many results, generally divided into specific installations (with some states appearing often), underlying policy discussions, and requirements.

One source of information is https://www.fhwa.dot.gov/publications/research/safety/04091/12.cfm#c1212, which observes in part that

> “As a rule of thumb, dual left-turn lanes are generally considered when left-turn volumes exceed 300 vehicles per hour (assuming moderate levels of opposing through traffic and adjacent street traffic). A left-turn demand exceeding 600 vehicles per hour indicates a triple left-turn may be appropriate”.
>
> “A study of double and triple left-turn lanes in Las Vegas, NV, showed that about 8 percent of intersection-related sideswipes occur at double lefts, and 50 percent at triple lefts …. These sideswipes are 1.4 and 9.2 percent of all crashes at the intersections with double and triple lefts, respectively. Turn path geometry and elimination of downstream bottlenecks are important considerations for reducing sideswipes”.

With regard to Part “a” of the assigned problem, the answer is that triple left turns are allowed in a number of jurisdictions and are cited in FHWA and other documents.

With regard to Part “b” of the assigned problem, the literature from the web search will provide many examples of installations and experience. The student is to be cautioned that some of the material is quite old (e.g. circa 1995) and will not have the benefit of the intervening decades of field experience. Better that the student focus on the last 5-10 years.

With regard to capacity of the 3rd lane, there is presently no unambiguous statement or rule of thumb that indicates that the 3rd lane adds "X" vehicles per hour, compared to the 2nd lane having added "Y" vehicles per hour of capacity, and so forth. Rather,

- The spirit of the literature current at the time of this writing is that "if the left turn demand cannot be handled in two left turn lanes, the 3rd lane will allow more vehicles in a shorter time, allowing more g/C for other phases".
- There are some simulation-based results of the capacity of triple left turn lanes, but there is also the wise observation that "existing traffic models do not treat triple left turns explicitly and can therefore only offer an approximation of their operation"[3]

The authors concur with that observation: simulation models are excellent tools, but do depend upon the "internal rules" that are built into them. Some of these rules are based upon empiric results, and some on modeling of specific aspects of traffic operations (car-following, discharge headways, allocation across lanes, gap acceptance, etc.). But the authors know of no specific rules built in for the rather special case of the 3rd left turn lane.

With regard to Part "c" of the assigned problem, guidelines do exist in some states and will be identified in the web search related to Part "a".

Before leaving the subject, the authors would ask the question, "Why?". That is, why is a triple left turn needed, even if it can exist? It would seem that

- There is a concentration of demand at a single intersection (or major driveway that is signalized) that cannot be moved elsewhere, despite best efforts;
- But moving some of it elsewhere --- for instance, a second entrance or driveway --- would seem to be a preferred choice, if at all feasible;
- And concentrating the demand into fewer vehicles --- higher occupancy autos, some bus traffic --- or moving some of it to other hours (peak spreading) would also seem to be a preferred choice, given triple left turns are still unfamiliar to many drivers and do consume space.

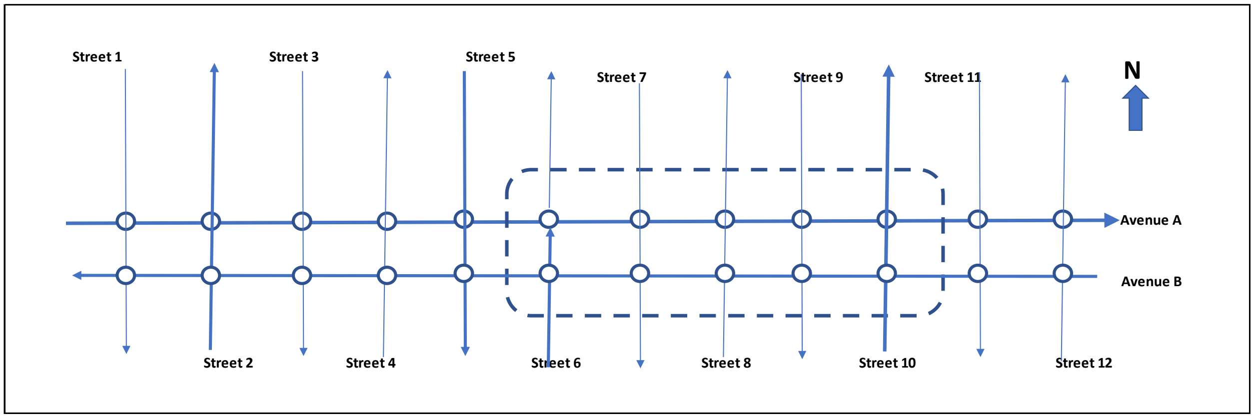
But it may well be that the receiving location does not have the alternative entrances, cannot impose higher occupancy policies, and needs the arrivals in a limited time window. That ultimately is a matter of professional judgment, but deserves consideration.

[3] http://www.fdot.gov/research/completed_proj/summary_te/fdot_bc131rpt.pdf

Problem 16-4

With regard to Part "a", the authors find the argument very plausible. While one can hope that intersections stay clear and there is no spillback that affects cross street traffic, that is not often the case without intensive on-the-ground enforcement. And spillback does cause congestion and oversaturation to spread like wildfire.

Consider the figure shown below, with the dashed area defining "the box" in which flow is to be maintained and with the thickness of the lines indicating relative traffic volumes:



Let us assume that on a good day, traffic moves without excessive numbers of stops, spillback, or delay …. but that a few intersections approach v/c of 0.90. In short, things are fragile.

We could consider a number of hypothetical events: (1) the intersection of Avenue A & Street 10 appears to be a critical intersection, and a likely source of problems; (2) Street 6 traffic must have a significant turning movement onto Avenue A, given the relative volumes along Street 6; (3) both Streets 10 and 6 can block Avenue B if there is spillback due to problems their traffic has at Avenue A; (4) Streets 5 and 2 have significant flows, even if they are outside the defined box; (5) an incident or event at Streets 11 or 12 & Avenue A can cause backups on Avenue A.

As we say, fragile:

- If problems occur inside the box, traffic can quickly start backing up, blocking or impeding some cross streets; in turn, at least two streets can soon affect Avenue B;
- The Avenue A backup can easily spread to upstream, affecting both its own upstream and the Street 5 cross traffic; problems can cascade west of the box, and be no respecter of our definition of "the box";
- For totally unknown reasons elsewhere in the network, Street 10 traffic demand may increase, affecting the most critical intersection. The same is true of other arriving demand, and of Avenue A capacity even east of Street 12;

- Or it can rain, reducing capacity everywhere by 5% or 10% and thereby driving up the v/c ratios everywhere;
- And we could go on.

But the key point is that while servicing the box as a priority, it is also quite important to not let the box fail, lest traffic congestion spread outside the box, notably upstream on Avenue A and then along Streets 5 and 2.

So, the authors accept that modest problems on Avenue A, upstream (that is, west) of the box, are acceptable to avoid a cascading effect that will affect this area much more significantly. One measure might be access management into the box, which means using the Avenue A upstream as "moving storage" --- greater queues, perhaps more stops, certainly more delay, and perhaps less g/C (and perhaps even more g/C for the streets west of the box.

Still, it is likely that the community and even drivers will notice the effects, and probably perceive that "we" are being impacted so that "they" inside the box can be served.

So, how creditable is the line that "Trust me, if we did not do that, it would have been even worse for you?". How do we show the effects of something *that did not happen*? Aside from letting it happen some of the time (which has practical and even ethical issues), perhaps simulation and public information --- starting with the media and local community boards or groups --- can help.

But the measured traffic is under the condition that something has been done. It will be a challenge to reconstruct the traffic pattern that did not happen. A starting point would be (a) documenting existing problems inside the box, even with the lower demand that is allowed in, because the capacity has been adversely affected[4], (b) using the extent of the "moving storage" upstream so that the performance of the box with lower capacity and original demand can be estimated --- as can its breakdown and promulgation upstream, with consequent adverse effects.

Of course, simulation sometimes is not as credible as real-world results, and constructing the "what if" traffic flows tends to look at circular (or self-serving) reasoning. Therefore, building trust with such groups by advance education and openness is quite important.

[4] That is, the v/c might have been 0.85 but a 10% decrease in capacity means it is 0.85/0.90 = 0.94. A reduction in arriving traffic by 10% would return the v/c to 0.85. But we did not say a reduction in true demand --- we said a reduction in arriving traffic, because the difference is placed in moving storage along Avenue A upstream of the box.

Problem 16-5

Regarding Part "a", the MUTCD can be downloaded by the student at no cost and stored as a PDF. The textbook encourages it, and it should have been done. But if not, it can be done now.

A simple search on "flashing yellow" just within the MUTCD PDF provides two full screens of results. Some are not germane to this problem assignment, but many are. In addition to the words, MUTCD Figures 4D-7, -12, and -14 show field configurations.

Regarding Part "b", the student should go to https://mutcd.fhwa.dot.gov/, look at the sidebar, and go to such topics as

- Interim Approvals
- Official Rulings
- Interpretations Issued by FHWA

It needs to be remembered that an "interim approval" can cease to exist for different reasons, including it is no longer interim but rather included in an official revision. The student will have to consider this, in years beyond the time of this writing.

Problem 16-6

Another good web search.

It should be emphasized to the students that this is not merely a performance goal --- it has implications for liability on the part of the state or local jurisdiction, if there is a crash that falls outside the allowed time window for remedy, following notification.

Two of the authors were involved in a training course for a group of state engineers, and the introductory remarks and welcoming was to be by the state traffic engineering chief. He welcomed them on that Monday morning by pointing out that a signal outage was reported on the prior Friday, nothing was done as of Sunday, and a person was killed in a crash at that intersection. He suggested that they had better pay attention in the short course, and then handed the group over to us.

Solutions to Problems in Chapter 17

Fundamentals of Intersection Design

Problem 17-1
The demands shown are clearly for a T-intersection. Some general observations for this intersection include:

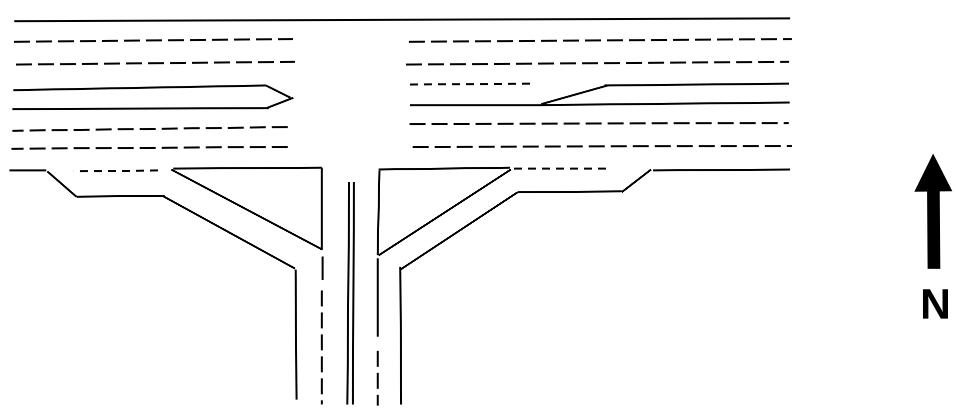
- Given the high turning flow rates, it would be desirable to provide an exclusive turning lane for all turning movement.

- Given the demand flow rates, it is likely that the intersection will be signalized, and that a 3-phase signal plan would be used.

- From Table 17-2, for a 3-phase signal, the through movements across the T will required 3 lanes each, and a cycle length of 90 s or more.

- If the 1200 veh/h movement can be permitted to move continuously, and extensive channelization used, it might be possible to use a 2-phase signal plan. There would have to be no pedestrians for this to be a viable option.

A design assuming a 3-phase signal plan is illustrated below.



Signal Phasing: Phase A: WB TH/LT
Phase B: EB/WB TH
Phase C: NB LT

Note: EB RT may run continuously with no control.
NB RT may run continuously; merge controlled by YIELD.

If it is possible to run the WB TH movement continuously, without pedestrians in the intersection, then a design similar to that in Figure 17-13 would be adopted, although the channelized right turns would be retained, and there would be three through lanes EB and WB.

Problem 17-2

Again, any design should start by considering the pattern and size of movements to be served. If we assume NB is the upward direction in the problem statement, the following observations can be made:

- This will doubtless be a signalized intersection.
- There are two very heavy LTs – EB and NB. The SB LT is sizeable, but much lower, and the WB LT is small.
- The SB and WB RTs are also very heavy.

The EB RT and NB LT are reciprocal movements, as are the SB RT and WB LT. All of these are very large movements. If right-of-way is available, strong consideration should be given to providing separate roadways for these movements, which will create a secondary intersection, which may itself have to be signalized. Removing these movements from the primary intersection will greatly simplify the primary intersection.

Given the size of the other movements, with hopefully two-phase signalization at the primary and secondary intersections, Table 17-2 suggests that three through lanes would be required for the E-W artery, while one might suffice for the N-S artery (two might be preferable, particularly if it could be dropped further downstream.

The figure at the end of this solution set illustrates a potential design for this situation.

Problem 17-3

A continuous green across the top of a signalized T-intersection is possible only if there are no pedestrians crossing the artery, or overpasses/underpasses provided for their use.

Problem 17-4

Offset intersections most often occur because developments on two sides of an arterial occurred at different times. Builders, in trying to optimize their use of lane, place streets to their advantage. Absent strong zoning laws and oversight, intersections on both sides of the street may not “line up” as traditional intersections, resulting in offsets.

Many techniques may be applied to help control such intersections:

- Where land is available, re-alignment to eliminate the offset at the intersection is the most desirable approach. Unfortunately, it is often not possible.

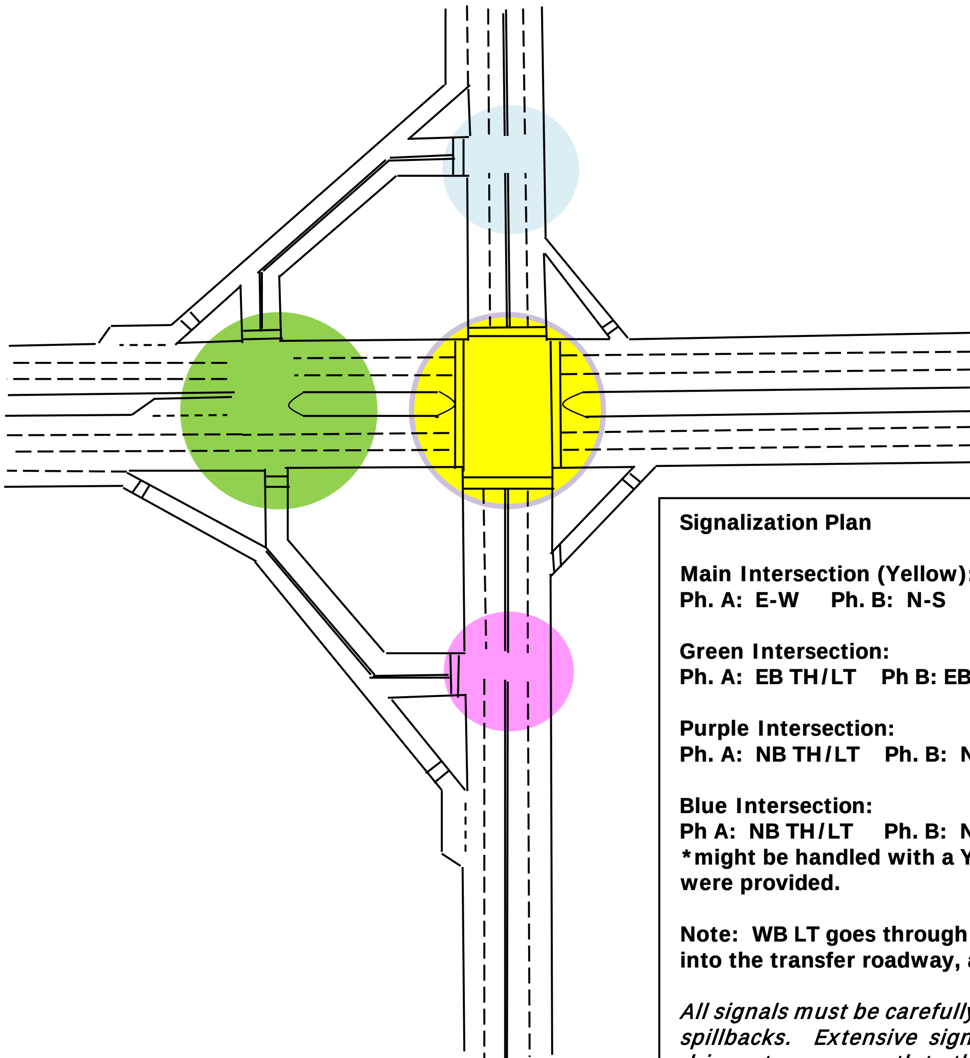
- Exclusive LT phasing for the non-aligned arterial can minimize LT conflicts with opposing through vehicles. Room for exclusive LT lanes must be available.

- Trajectory markings through the intersection are used to help vehicles navigate a safe path across the offset.

- Pedestrian paths must be clearly marked by crosswalks, with accompanying pedestrian signals and signs.

- Signal heads should be properly arranged so that vehicles (and pedestrians) can see them and clearly interpret which movements and lanes they control.

Drawing for Solution to Problems 17-2



Signalization Plan

Main Intersection (Yellow):
Ph. A: E-W Ph. B: N-S No turns permitted.

Green Intersection:
Ph. A: EB TH/LT Ph B: EB/WB TH Ph. C: NB/SB LT

Purple Intersection:
Ph. A: NB TH/LT Ph. B: NB TH, SB Ph. C: EB LT

Blue Intersection:
Ph A: NB TH/LT Ph. B: NB TH, SB Ph. C: EB RT*
***might be handled with a YIELD sign if a merge lane were provided.**

Note: WB LT goes through the intersection, makes RT into the transfer roadway, and another RT at its end.

All signals must be carefully coordinated to avoid queue spillbacks. Extensive signing must be used to guide drivers to proper path to their desired destination.

Solutions to Problems in Chapter 18

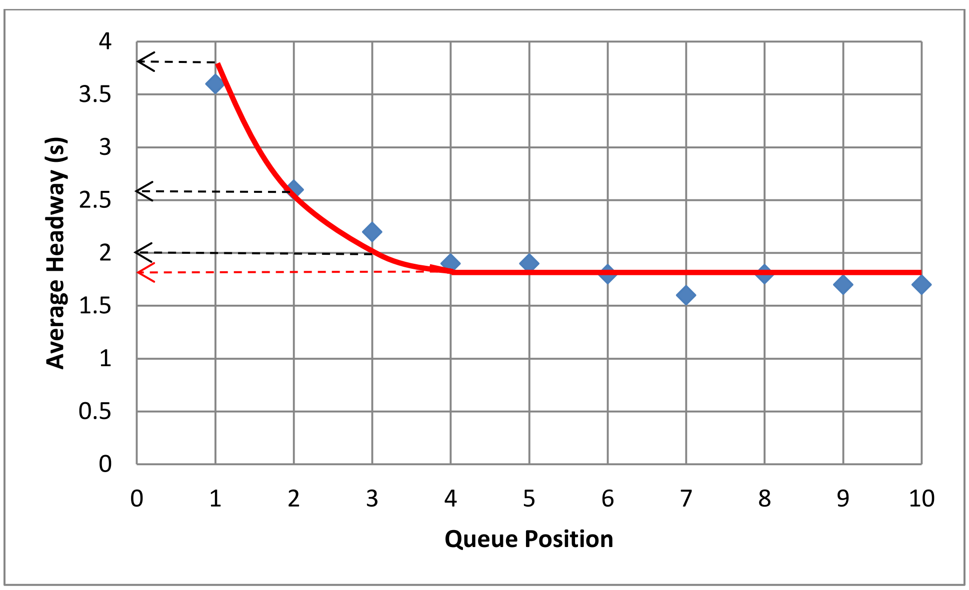
Principles of Intersection Signalization

Problem 18-1

For each queue position (1-10), a spreadsheet is used to compute the average headway. Note that in computing the average headway, the number of observations differs depending upon the number of queues that had a vehicle in that position. For positions 1-8, there are 10 headways each. Position 9 has 5 headways, and Position 10 has only 4 headways.

A plot of average headway vs. queue position is plotted, from which the saturation headway can be extracted as shown.

Q Position	C1	C2	C3	C4	C5	C6	C7	C8	C9	C10	TOT	No of Hdws	Avg Hdwy
1	3.6	3.7	3.5	3.6	3.4	3.3	3.6	3.7	3.5	3.7	35.6	10	3.6
2	2.6	3.0	2.4	2.6	2.2	2.2	2.7	2.8	2.7	2.8	26.0	10	2.6
3	2.0	2.4	2.0	2.1	1.8	2.0	2.4	2.4	2.3	2.4	21.8	10	2.2
4	1.7	2.0	2.0	2.0	1.7	2.0	2.0	2.0	2.0	2.0	19.4	10	1.9
5	1.6	1.9	2.0	1.9	1.7	1.8	1.9	2.1	2.0	1.8	18.7	10	1.9
6	1.7	1.8	1.9	1.9	1.6	1.7	1.8	1.8	1.8	1.7	17.7	10	1.8
7	1.7	1.8	1.8	1,8	1.7	1.7	1.9	1.9	1.9	1.9	16.3	10	1.6
8	1.6	1.7	1.8	1.8	1.7	1.7	1.9	1.8	1.9	1.9	17.8	10	1.8
9		1.8		1.6		1.7	1.7			1.7	8.5	5	1.7
10		1.7		1.8			1.7			1.7	6.9	4	1.7



Fitting the curve by eye is something of an art. In general, it is assumed that headways after the 4th position tend to level out. Thus, a horizontal line that best fits the average headway of positions 4 through 10 is first drawn. Then, a smooth curve that connects with this straight portion of the curve is fit. Distances of points above and below the fit lines should more or less balance.

(a) From the curve, the following values may be determined:

The saturation headway, h, is read as the extension of the flat portion of the curve, or **1.8 s/veh**.

The start-up lost time, ℓ_1, is the difference between the actual headways (as defined by the curve, not the points) and the saturation headway for the first three headways, or:

3.8 – 1.8	=	2.0
2.6 – 1.8	=	0.8
2.0 – 1.8	=	0.2
TOTAL		**3.0 s/phase**

(b) The saturation flow rate is computed as:

$$s = \frac{3600}{h} = \frac{3600}{1.8} = 2{,}000 \; veh/hg/\ln$$

Problem 18-2

The capacity of a signalized intersection approach is computed as:

$$c = s * \left({}^{g}\!/\!_{C} \right)$$

where:

where:	*s*	=	3600/h = 3600/2.25 = 1,600 veh/hg/ln
	g	=	G + Y - $\ell_1 - \ell_2$ = 50+4.0-2.0-1.5 = 50.5 s
	C	=	90 s (given)

Then:

$$c = 1600 * \left({}^{50.5}\!/\!_{90} \right) = 898 \; veh/h/\ln$$

As the approach has 2 lanes, the approach capacity is 2 x 898 = 1,796 veh/h.

Problem 18-3

The maximum sum of critical lane volumes is computed as:

$$V_c = \frac{1}{h}\left[3600 - N\,t_L\left(\frac{3600}{C}\right)\right]$$

$$V_c = \frac{1}{2.2}\left[3600 - 3*4*\left(\frac{3600}{90}\right)\right] = \frac{3600-480}{2.2} = 1{,}418\ veh/h$$

Problem 18-4

The solution begins by determining the maximum sum of critical lane volumes that can be accommodated by the signalization described:

$$V_c = \frac{1}{h}\left[3600 - N\,t_L\left(\frac{3600}{C}\right)\right]$$

$$V_c = \frac{1}{2.4}\left[3600 - 2*4*\left(\frac{3600}{120}\right)\right] = \frac{3600-240}{2.4} = 1{,}400\ veh/h$$

The existing critical volumes are 1,000 tvu/h NB and 2,000 tvu/h WB.

The listing below shows the sum of critical lane volumes for various lane scenarios:

NB Lanes	SB Lanes	Sum of Critical Lane Volumes
1	1	1000 + 2000 = 3,000 tvu/h > 1400 NG
1	2	1000 + 2000/2 = 2,000 tvu/h > 1400 NG
2	2	1000/2 + 2000/2 = 1,500 tvu/h > 1400 NG
2	3	1000/2 + 2000/3 = 1,167 tvu/h < 1400 OK
3	3	1000/3 + 2000/3 = 1,000 tvu/h < 1400 OK

The minimum design would call for 3 lanes EB and WB, and two lanes NB and SB.

Problem 18-5

For the case cited, the sum of critical lane volumes (V_c) is 1,000 tvu/h. Then:

(a) The absolute minimum cycle length that could be used is computed as:

$$C_{\min} = \frac{N\,t_L}{1-\left(\frac{V_c}{3600/h}\right)} = \frac{2*4}{1-\left(\frac{1000}{3600/2.4}\right)} = \frac{8}{1-0.667} = \frac{8}{0.333} = 24.0\ s,\ \ SAY\ \mathbf{30}\ s$$

The absolute minimum cycle length, however, provides for no unused green time, and assumes no variation of traffic within the analysis hour.

(b) The desirable cycle length accounts for the PHF and for some unused green time in the cycle:

$$C_{des} = \frac{N\,t_L}{1-\left[\frac{V_c}{(3600/h)*PHF*(v/c)}\right]} = \frac{2*4}{1-\left[\frac{1000}{(3600/2.4)*0.98*0.92}\right]} = \frac{8}{1-0.739} = 30.65\ SAY\ 35\ s$$

Problem 18-6

(a) The through-vehicle equivalent is found as:

$$40 = 10 + 20\,E_{LT}$$
$$E_{LT} = \frac{40-10}{20} = 1.5$$

(b) For the case described, 20 out of 40+10+20 = 70 vehicles, or 20/70 = 0.2857 of the vehicles are turning left. Then:

$$f_{LT} = \frac{1}{1+P_{LT}\left(E_{LT}-1\right)} = \frac{1}{1+0.2857(1.5-1)} = \frac{1}{1.14285} = 0.875$$

Problem 18-7

Left-turns:	800*0.20*2.5 =	400 tvu's
Through & RTs:	800*0.80*1.0 =	640 tvu's
Total		**1,040 tvu's**

Problem 18-8

(a) The start-up lost time in the equation is the first term, or 2.04 s/phase.

(b) The saturation headway suggested by the equation is 2.35 s/veh, which produces a saturation flow rate of 3600/2.35 = 1,531.9, say 1,532 veh/hg.

Problem 18-9

Left turns:	1350*0.08*2.7	=	292 tvu's
Through & RTs:	1350*0.92*1.0	=	1,242 tvu/s
Total:			**1,534 tvu's**

Problem 18-10

The capacity of the approach is:

$$c = s * \left(\frac{g}{C} \right) = 1{,}450 * \left(\frac{50}{80} \right) = 906 \ veh/h$$

The v/c ratio is, therefore, 500/906 = 0.552. This value is quite low. For a v/c ratio this low, Webster's Delay Equation is appropriate for use in predicting delay:

$$UD = \frac{0.50 * C * \left[1 - \left(\frac{g}{C} \right) \right]^2}{1 - \left[\left(\frac{g}{C} \right) * X \right]} = \frac{0.50 * 80 * [1 - (50/80)]^2}{1 - [(50/80) * 0.552]} = 8.6 \ s/veh$$

Problem 18-11

Operating with a v/c ratio of 1.05, this intersection experiences both uniform and overflow delay. In this v/c range, the v/c ratio is best estimated using Akcelik's equation.

In order to make these computations, the capacity and saturation flow rate for the intersection must be computed. This is done using the given demand flow rate, and the known v/c ratio:

$$v/c = 1.05 = \frac{800}{c}$$
$$c = 800/1.05 = 762 \ veh/h$$
$$c = 762 = s * \left(\frac{g}{C} \right) = s * 0.60$$
$$s = 762/0.60 = 1{,}270 \ veh/hg$$

Uniform delay is estimated using Webster's Delay Equation for the simplified case in which v/c = 1.00. Remember that the maximum v/c ratio that can be used in Webster's Equation is 1.00:

$$UD = 0.50\,C[1 - (g/C)] = 0.50 * 75 * (1 - 0.60) = 15.0\, s/veh$$

This delay applies to any time period for which the demand situation is as stated. Akcelik's equation can be used to estimate the overflow delay that occurs over the ½ hour, or over the first 5 minutes of the hour. Note that the saturation flow rate is expressed as veh/s in Akcelik's equation:

$$OD = \frac{cT}{4}\left[(X-1)+\sqrt{(X-1)^2+\left(\frac{12(X-X_o)}{cT}\right)}\right]$$

$$X_o = 0.67+\left(\frac{sg}{600}\right)=0.67+\left(\frac{(1270/3600)*(75*0.60)}{600}\right)=0.6965$$

$$OD_{30\min}=\frac{760*0.50}{4}\left[(1.05-1)+\sqrt{(1.05-1)^2+\left(\frac{12*(1.05-0.6965)}{760*0.50}\right)}\right]=15.9\ s/veh$$

$$OD_{5\min}=\frac{760*0.0833}{4}\left[(1.05-1)+\sqrt{(1.05-1)^2+\left(\frac{12*(1.05-0.6965)}{760*0.0833}\right)}\right]=5.0\ s/veh$$

The total delay is the sum of the uniform delay and the overflow delay:

$$d_{30}=15.0+15.9=30.9\ s/veh$$
$$d_5=15.0+5.0=20.0\ s/veh$$

Even though there is an overflow situation, the delays are not very high because of two reasons: (a) the demand is only 5% higher than the capacity, and (b) the condition exists for only ½ hour. Delay in the first five minutes of overflow is obviously less than the average over ½ hour, as the queue has not yet fully developed.

<u>Problem 18-12</u>

To determine the most appropriate equation for use in predicting delay, the v/c ratio for the hour should be considered:

$$c = s*\left(g/C\right)=3250*\left(55/100=1{,}788\ veh/hg\right)$$
$$v/c = 2{,}000/1{,}788=1.12$$

This value is significantly higher than 1.00. In this case, the simple theoretical equations for overflow delay may be employed. As in the previous example (18-11), the simplified equation for Uniform Delay (Webster's Equation) can be used. This value applies to all time periods during which the stated conditions exist:

$$UD = 0.50C\left[1-\left(g/C\right)\right]=0.50*100*(1-0.55)=22.5\ \ s/veh$$

(a) Overflow delay for the full hour is computed as:

$$OD=\frac{T}{2}(X-1)=\frac{3600}{2}(1.12-1)=216\,s/veh$$

where X = v/c = 1.12, and 3600 is the number of seconds in an hour.

(b) The same formula is used to estimate the overflow delay during the first 15 minutes of the hour. In this case, T = 60*15 = 900 s:

$$OD = \frac{900}{2}(1.12 - 1) = 54\, s/veh$$

(c) The overflow delay in the last 15 minutes of the hour is estimated as:

$$OD = \frac{T_1 + T_2}{2}(X - 1)$$

where T_1 = 45*60 = 2,700 s, and T_2 = 60*60 = 3,600 s:

$$OD = \frac{2700 + 3600}{2}(1.12 - 1) = 378\, s/veh$$

Total delays must add the UD to OD to obtain:

$$d_{hour} = 22.5 + 216.0 = 238.5\ s/veh$$
$$d_{first15} = 22.5 + 54.0 = 76.5\ s/veh$$
$$d_{last15} = 22.5 + 378.0 = 400.4\ s/veh$$

Obviously, delay during the first 15 minutes is less severe than delay in the last 15 minutes, as there is no residual queue at T = 0, and there is a substantial residual queue at T = 45 minutes.

Solution to Problems in Chapter 19

Fundamentals of Signal Timing and Design – Pretimed Signals

Problem 19-1
All of the left turns require protection, for the following reasons:

Intersection 1, EB:	Opposing S_{85} = 50 + 5 = 55 mi/h > 45 mi/h (Table 19-1)
Intersection 1, WB:	Opposing S_{85} = 50 + 5 = 55 mi/h > 45 mi/h (Table 19-1)
Intersection 2, EB:	V_{LT} = 250 veh/h > 200 veh/h (Eqn 19-1) *xprod* = 250*1800/3 = 150,000 > 50,000 (Eqn 19-2) No. of LT lanes = 2 ≥ 2 (Table 19-1) LT crashes = 18 (both directions)/3 yrs > 13 (Table 19-2)
Intersection 2, WB:	V_{LT} = 250 veh/h > 200 veh/h (Eqn 19-1) *xprod* = 250*1800/3 = 150,000 > 50,000 (Eqn 19-2) No. of LT lanes = 2 ≥ 2 (Table 19-1) LT crashes = 18 (both directions)/3 yrs > 13 (Table 19-2)
Intersection 3, EB:	Opposing S_{85} = 50 + 5 = 55 mi/h > 45 mi/h (Table 19-1)
Intersection 3, WB:	Opposing S_{85} = 50 + 5 = 55 mi/h > 45 mi/h (Table 19-1)

Problem 19-2
The change interval is computed using Equation 19-3:

$$y = t + \frac{1.47\,S_{85}}{2(a + 32.2\,G)}$$

where:

S_{85}	=	35 + 5 = 40 mi/h
t	=	1.0 s
a	=	10 ft/s^2
G	=	-2/100 = -0.02

$$y = 1.0 + \frac{1.47 * 40}{2[10 + (32.2 * -0.02)]} = 1.0 + \frac{58.8}{18.712} = 4.14\ s \approx 4.1\ s$$

The clearance interval is given by Eqn 19-5 for significant pedestrian movements:

$$ar = \frac{P + L}{1.47\ S_{15}}$$

where:

S_{15}	=	35 – 5 = 30 mi/h
P	=	50 + 10 + 2 = 62 ft
L	=	20 ft

$$ar = \frac{62+20}{1.47*30} = 1.86\,s \approx 1.9\,s$$

<u>Problem 19-3</u>

The analysis of this situation depends upon which pedestrian crossing policy is in effect. To summarize:

Option 1: pedestrians may be in crosswalk during *G, y,* and *ar* intervals.
Option 2: pedestrians may be in crosswalk during *G* and *y* intervals.
Option 3: pedestrians may be in crosswalk during *G* interval only.

Each phase is tested vs. each of these options:

Phase A: Option 1 – *G+y+ar* = 21.5 + 3.0 + 1.5 = 26.0 s < 30.0 s *NG*
Option 2 – *G + y* =21.5 + 3.0 = 24.5 s < 30.0 s *NG*
Option 3 – *G* = 21.5 < 30.0 s *NG*

Phase B: Option 1 – *G+y+ar* = 60.0 + 3.0 + 1.0 = 64.0 > 15.0 s *OK*
Option 2 – *G+y* = 60.0 + 3.0 = 63.0 > 15.0 s *OK*
Option 3 – *G* = 60.0 > 15.0 s *OK*

Phase A is not safe for pedestrians under any of the three policies, while Phase B is safe for any of the three policies.

To insure that pedestrians on both phases are safely accommodated in every signal cycle, the green interval (*G*) for Phase A must be increased to:

Option 1: 30.0 – 3.0 – 1.5 = 25.5 s
Option 2: 30.0 – 3.0 = 27.0 s
Option 3: 30.0 s

This must be done in a way that keeps the current balance of green times between the two vehicular phases. Note that the current cycle length is 90s (the sum of all G, y, and ar intervals). The cycle length would be increased to:

Option 1: 90*(25.5/21.5) = 106.7 s, *say* 110 s
Option 2: 90*(27.0/21.5) = 113.0 s, *say* 120 s
Option 3: 90*(30.0/21.5) = 125.6 s, *say* 130 s

For a pretimed signal, Option 3 results in an unreasonably high cycle length. Options 1 and 2 result in cycle lengths in the acceptable range of normal use. The green times for Phases A and B would be allocated in the same ratio as the exiting timing: 21.5 (Phase A) to 60 (Phase B). As the *y* and *ar* intervals total 8.5 s for the two phases, the remaining time is green. The retimed green times would be:

Option 1:

$$G_A = (110 - 8.5) * \left(\frac{21.5}{21.5 + 60.0} \right) = 26.8\,s$$
$$G_B = (110 - 8.5) * \left(\frac{60.0}{21.5 + 60.0} \right) = 74.7\,s$$

Option 2:

$$G_A = (120 - 8.5) * \left(\frac{21.5}{21.5 + 60.0} \right) = 29.4\ s$$
$$G_B = (120 - 8.5) * \left(\frac{60.0}{21.5 + 60.0} \right) = 82.1\ s$$

These re-timings would provide safety for pedestrians under Options 1 and 2. Option 3 would not normally be considered, as it yields a cycle length outside the normal range of use.

Problem 19-4

The intersection for Problem 19-4 is quite simple: two one-way streets. There are no opposed turns, but each of the legal turns has an exclusive lane that must be taken into account.

Step 1: Develop a Phase Plan

The phase plan in this case is quite simple, as there are no opposed turns to consider. A simple two-phase plan will be adopted, with Phase A assigned to the NB approach and Phase B assigned to the WB approach. (These could be reversed, as the order does not matter.)

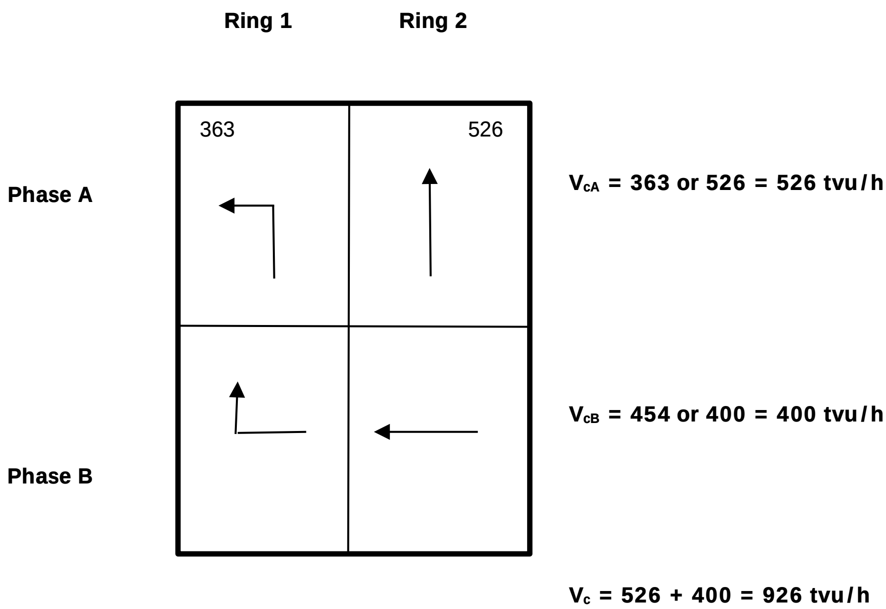
Step 2: Convert Volumes to tvu's

Equivalents are given in Tables 19-4 for LTs and 19-5 for RTs. In this case, we have a unique situation: the NB LT is unopposed. It could be treated as a *protected* LT with an equivalent value of 1.05, a *permitted* LT with an equivalent of 1.10, or a right turn through a low pedestrian flow, with an equivalent of 1.21 (from Table 19-5). The latter approach will be taken here, as the interaction between the LT and pedestrians in the crosswalk is virtually analogous to a RT.

Movement	Approach	Volume (veh/h)	Equivalent (T 19-4/5)	Volume (tvu/h)	Lane Group Vol (tvu/h)	No. of Lanes	Volume/Ln (tvu/h)
LT	NB	300	1.21	363	363	1	363
TH		2105	1.00	2105	2105	4	526
TH	WB	1200	1.00	1200	1200	3	400
RT		375	1.21	454	454	1	454

Step 3: Determine the Sum of Critical Lane Volumes

A ring diagram for the proposed signal timing is shown below, with the critical lane volumes (tvu/h) shown in the appropriate cells. Note that due to the existence of exclusive turn lanes on each approach, both rings are used in the signalization.



Step 4: Determine Yellow and All-Red Intervals

The length of the *yellow* intervals is computed using Equation 19-3. Note that the S_{85} for the NB approach is 40 + 5 = 45 mi/h, and for the WB approach is 35 + 5 = 40 mi/h. Then:

$$y = t + \frac{1.47\,S_{85}}{2(a + 32.2G)}$$

$$y_A = 1.0 + \frac{1.47 * 45}{2(10 + 32.2 * 0)} = 3.6\ s$$

$$y_B = 1.0 + \frac{1.47 * 40}{2(10 + 32.2 * 0)} = 3.3\ s$$

The length of the *all-red* intervals is computed using Equation 19-4 (for low pedestrian volumes). Note that the S_{15} is 40 – 5 = 35 mi/h for the NB approach and 35 – 5 = 30 mi/h for the WB approach. Then:

$$ar = \frac{w + L}{1.47\,S_{15}}$$

$$ar_A = \frac{60 + 18}{1.47 * 35} = 1.9\ s$$

$$ar_B = \frac{55 + 18}{1.47 * 30} = 2.0\ s$$

Step 5: Determine the Lost Time Per Cycle

Because the usual default values for ℓ_1 and *e* (both 2.0 s) are in place, the total lost time per cycle is the sum of the *yellow* and *all-red* intervals in the cycle:

$$L = (3.6 + 1.9) + (3.3 + 2.0) = 10.8\ s$$

Step 6: Determine the Desirable Cycle Length
The desirable cycle length is computed using Equation 19-11:

$$C_{des} = \frac{L}{1 - \left[\dfrac{V_c}{1700\, PHF\, (v/c)} \right]}$$

$$C_{des} = \frac{10.8}{1 - \left[\dfrac{926}{1700 * 0.90 * 0.95} \right]} = \frac{10.8}{1 - 0.637} = 29.75\ s,\ \ say\ 30\ s$$

Step 7: Splitting the Green
Equation 19-13 is used to split the effective green time in the cycle. The amount of effective green time in the cycle (g_{TOT}) is 30-10.8 = 19.2 s. Then:

$$g_i = g_{TOT} \left(V_{ci} / V_c \right)$$
$$g_A = 19.2 \left(526/926 \right) = 10.9\ s$$
$$g_B = 19.2 \left(400/926 \right) = 8.3\ s$$

Note that due to the use of standard default values for ℓ_1 and *e*, actual green times are equal to effective green times.

Step 8: Check Pedestrian Requirements
The minimum crossing time required by pedestrians is given by Equation 19-15:

$$G_{pi} = PW_i + PC_i$$

From Table 19-6, for low pedestrian volumes, the minimum *PW* is 4.0 s.

The required pedestrian clearance time (*PC*) is given by Equation 19-16:

$$PC_i = L_i / S_p$$
$$PC_A = 60/4.0 = 15.0\, s$$
$$PC_B = 55/4.0 = 13.8\ s$$

Thus:

$$G_{pA} = 4.0 + 15.0 = 19.0\,s$$
$$G_{pB} = 4.0 + 13.8 = 17.8\,s$$

If Option 1 is in effect, allowing pedestrians to be in the crosswalk during *green*, *yellow*, and *all-red* intervals, then the actual time provided for pedestrians is:

Phase A: 10.9 + 3.6 + 1.9 =16.4 s < 19.0 s *NG*
Phase B: 8.3 + 3.3 + 2.0 = 13.5 s < 17.8 s *NG*

Obviously, if Option 1 does not work, then Options 2 and 3 would be worse. The cycle length would have to be increased to provide for vehicular green times that are adequate for pedestrians during every signal cycle. This could be done for every option, but a solution is shown here only for Option 1.

The Phase A green time of 10.9 s would have to be increased to 19.0 – 3.6 – 1.9 =13.5 s, an increase of 13.5/10.9 = 1.24. The Phase B green time of 8.3 s would have to be increased to 17.8 - 3.3 – 2.0 = 12.5 s, an increase of 12.5/8.3 = 1.51. Phase B requires the larger increase. Therefore, the new cycle length would have to be:

$$C_{new} = 30.0 * 1.51 = 45.3\,s,\ say\ 50\ s.$$

Green times are now reallocated using Equation 19-13. In a 50-s cycle, there is 50.0-10.8 = 39.2 s of effective green time to allocate (g_{TOT}). Then:

$$g_A = 39.2\left(\frac{526}{926}\right) = 22.2\ s$$
$$g_B = 39.2\left(\frac{400}{926}\right) = 17.0\,s$$

Note that g_B was increased by 0.1 s to account for round-off errors. The total (22.2+17.2) *must* equal 39.2 s. As in the original solution, actual green times are equal to effective green times in this case.

Problem 19-5

The subject intersection shows a major arterial with left-turn lanes intersecting with a collector/arterial with one lane in each direction.

Step 1: Develop a Phase Plan

The phase plan depends upon whether or not any or all of the left turn movements at the intersection require protection. Each of the left turns is considered in sequence:

EB LT: V_{LT} = 200 veh/h ≥ 200 veh/h
(protection needed)

WB LT: V_{LT} = 160 veh/h < 200 veh/h
xprod = 160*(800/3) = 42,667 < 50,000
S_{85} = 40 + 5 = 45 mi/h = 45 mi/h
No accident information given.
(protection not needed)

NB LT: V_{LT} = 10 veh/h < 200 veh/h
xprod = 10*(400/1) = 4,000 < 50,000
S_{85} = 30 + 5 = 35 mi/h < 45 mi/h
No accident information given.
(protection not needed)

SB LT: V_{LT} = 12 veh/h < 200 veh/h
xprod = 12*(420/1) = 5,040 < 50,000
S_{85} = 30 + 5 = 35 mi/h < 45 mi/h
(protection not needed)

According to the analysis of general guidelines, only the EB LT needs a protected phase. There is, however, a separate LT lane for WB LTs, and there appears to be no apparent need for a EB TH/RT phase that is significantly longer than the WB TH/RT. Therefore, since 160 LTs/h is not an insignificant volume, and because we will be providing an EB LT phase, we will provide a simultaneous LT phase for the WB LTs. As the EB and WB LTs are not significantly different, an exclusive LT phase will suffice.

Step 2: Convert Volumes to tvu's

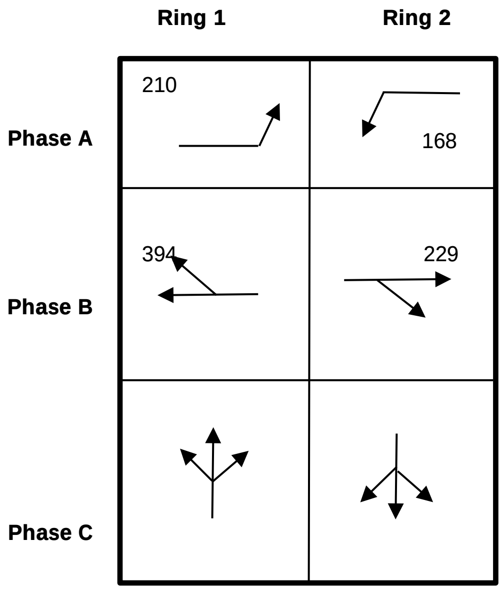
Through vehicle equivalent values are found in Tables 19-4 and 19-5. The conversions are shown in the table that follows:

Approach	Mvt	Volume (veh/h)	Equiv. (T19-4/5)	Volume (tvu/h)	Lane Grp Volume (tvu/h)	No. of Lanes	Vol/Ln (tvu/h)
EB	L	200	1.05	210	210	1	210
	T	800	1.00	800	958	3	329
	R	120	1.32	158			
WB	L	160	1.05	168	168	1	168
	T	1050	1.00	1050	1182	3	394
	R	100	1.32	132			
NB	L	10	2.50	25	458	1	458
	T	420	1.00	420			
	R	10	1.32	13			
SB	L	12	*2.70*	32	443	1	443
	T	400	1.00	400			
	R	8	1.32	11			

Italics indicate a value interpolated in Table 19-4.

Step 3: Determine the Sum of Critical Lane Volumes

A ring diagram for the signal phase plan is shown below with all of the lane volumes entered. The critical path is determined as the one which yields the highest sum of critical lane volumes.



V_{cA} **= 210 or 168** $\Rightarrow$ **210 tvu/h**

V_{cB} **= 394 or 229** $\Rightarrow$ **394 tvu/h**

V_{cC} **= 458 or 443** $\Rightarrow$ **458 tvu/h**

Step 4: Determine Yellow and All-Red Intervals

The *yellow* interval is determined by Equation 16-3, while the *all-red* will be determined using Equation 19-5, as there are moderate pedestrian flows at the intersection. Then:

$$y = t + \frac{1.47\,S_{85}}{2(a + 32.2G)}$$

$$y_{A,B} = 1.0 + \frac{1.47(40+5)}{2(10+32.2*0)} = 4.3\ s$$

$$y_C = 1.0 + \frac{1.47(30+5)}{2(10+32.2*0)} = 3.6\ s$$

$$ar = \frac{P+L}{1.47\,S_{15}}$$

$$ar_{A,B} = \frac{(30+10+2)+18}{1.47(40-5)} = 1.2\ s$$

$$ar_C = \frac{(85+10+2)+18}{1.47(30-5)} = 3.1\ s$$

Step 5: Determine the Lost Time Per Cycle (L)

Note that because the usual defaults for ℓ_1 and *e* are in place (both 2.0 s), that the lost time per cycle is equal to the sum of the *yellow* and *all-red* intervals. This is three-phase signal, with three sets of *yellow* and *all-red* intervals, two associated with the E-W artery, and one with the N-S cross street.

$$L=(4.3+1.2)+(4.3+1.2)+(3.6+3.1)=17.7\ s$$

Step 6: Determine the Desirable Cycle Length

The desirable cycle length is given by Equation 19-11:

$$C_{des}=\frac{L}{1-\left[\dfrac{V_c}{1700\,PHF\,(v/c)}\right]}$$

$$C_{des}=\frac{17.7}{1-\left[\dfrac{1062}{1700*0.92*0.95}\right]}=\frac{17.7}{1-0.715}=62.1\ s\Rightarrow 65\,s$$

Step 7: Split the Greens

The total amount of green time to be split is 65.0-17.7 = 47.3 s. Using Equation 19-13:

$$g_i=g_{TOT}\left(\frac{V_{ci}}{V_c}\right)$$

$$g_A=47.3\left(\frac{210}{1062}\right)=9.4\,s$$

$$g_B=47.3\left(\frac{394}{1062}\right)=17.5\ s$$

$$g_C=47.3\left(\frac{458}{1062}\right)=20.4\,s$$

Due to the use of the usual defaults, actual green times are equal to effective green times.

Step 8: Check Pedestrian Requirements

Note that pedestrians cross the N-S street during Phase B, and the E-W street during Phase C. Minimum pedestrian requirements are computed using Equation 19-15:

$$G_{Pi}=PW_i+PC_i$$

From Table 19-6, using the "typical" pedestrian volume category, the recommended minimum pedestrian WALK interval (*PW*) is between 7 and 10 s. Because the cycle length is close to the 60-sec boundary, we will use 7.0 s. Pedestrian clearance intervals (*PC*) are found using Equation 19-16:

$$PC = \frac{L}{S_p}$$

$$PC_B = \frac{30}{3.5} = 8.6\,s$$

$$PC_C = \frac{85}{3.5} = 24.3\ s$$

Thus:

$$G_{pB} = 7.0 + 8.6 = 15.6\ s$$

$$G_{pC} = 7.0 + 24.3 = 31.3\ s$$

The actual green time for Phase B is 17.5 s > 15.6 s. Therefore, Phase B is safe for pedestrians under any pedestrian policy. The actual green time for Phase C is 20.4 s < 31.3 s. Even if the *yellow* and *all-red* intervals are added (Option 1), the amount of pedestrian time provided is 20.4 + 3.6 + 3.1 = 26.8 s < 31.3 s.

Therefore, to accommodate pedestrians crossing the E-W street during Phase C, the cycle length must be increased. The retiming will assume that Policy 2 is in place, i.e., pedestrians may use the *yellow* interval, but NOT the *all-red* interval. The Phase C green must, therefore, be increased to 31.3 – 3.6 = 27.7 s, a ratio of 27.7/20.4 = 1.36. The new cycle length would, therefore, be 65*1.36 = 88.4 s $\Rightarrow$ 90 s. There are 90.0 – 17.7 = 72.3 s of effective green time within the 90-sec cycle to allocate using Equation 19-13:

$$g_A = 72.3\left(210/1062\right) = 14.3\ s$$

$$g_B = 72.3\left(394/1062\right) = 26.8\ s$$

$$g_C = 72.3\left(458/1062\right) = 31.2\ s$$

As previously, the actual green times are the same as the effective green times. This timing is safe for all pedestrians on all phases under the Option 2 policy.

To accomplish this requires that the vehicular cycle length be increased by almost a third from what vehicles needed. Delays to vehicles will increase somewhat. On the other hand, with moderate pedestrian volumes (approximately 200 peds/h/xwalk), a pedestrian actuator could not be considered, as it would be pushed on virtually every cycle, fully disrupting the vehicular timing plan.

The actual WALK phases (*PW*) will be larger than the minimum of 7.0 s needed. The pedestrian clearance ends (under Option 2) at the end of the *yellow* interval. Working backwards, the *PW* covers any remaining vehicular green time. Specifically:

$$PW_i = G_i + y_i - PC_i$$
$$PW_B = 26.8 + 4.3 - 8.6 = 22.5\ s$$
$$PW_C = 31.2 + 3.6 - 24.3 = 10.5\ s$$

Problem 19-6

The intersection has a distinctive geometry that must play a major role in developing a signal timing and design. The Y-intersection provides some unique vehicular conflicts, and presents a significant challenge for pedestrians. Note that there is *no crosswalk* on Main Street. This is because no matter what signalization is adopted, there will be a major flow at relatively high speed crossing the normal pedestrian path across Main Street in every phase. The only way to provide for safe passage (other than including a pedestrian-only phase) is to restrict crossings to First Street and Church Road. This may be inconvenient for pedestrians, but (fortunately) in this case the level of pedestrian activity is low.

Step 1: Develop a Phase Plan

There are really only two options for phasing.

A two-phase plan would be comprised of:

Phase A: Main Street NB Left and First Street SB; pedestrians may cross Church Road.

Phase B: Main Street NB Right and Church Road SB; pedestrians may cross First Street.

A three-phase plan would be comprised of:

Phase A: Main Street NB, all movements. No pedestrian movements permitted.

Phase B: First Street SB, all movements. Pedestrians may cross Church Rd.

Phase C: Church Road SB, all movements. Pedestrians may cross First Street.

The two-phase plan would allow some opposed left turns from First Street, but the volume of these is low. It would be more efficient than the three-phase plan.

The three-phase plan would be easier to comprehend for drivers, and there would be no opposed left turns. It would be less efficient, as it would involve more lost time and a longer cycle length.

The ultimate choice is between efficiency and clarity for drivers with some safety benefits. Either approach is reasonable given the demand volumes. The three-phase option is illustrated in this solution.

Step 2: Convert Volumes to tvu's

The unique geometry of the Y-intersection also influences this step of the process. There are no true "through" movements, but "turns" are also not those normally encountered. The NB movements, the First Street RT, and the Church Street LT are high-speed movements that can effectively be considered to be "through" movements, with an equivalent of 1.0.

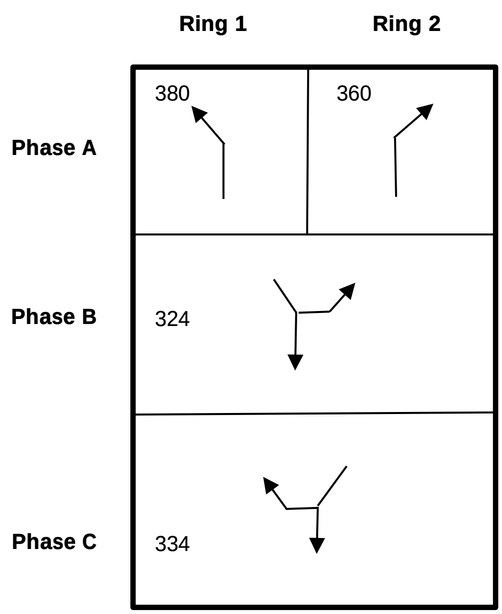
The First Street LT and Church Road RT are true turns that are even more difficult because they involve turning more than 90º. Neither has an opposing flow, so both will be treated as right turns with a conflicting pedestrian flow. From Table 19-5, with low pedestrian flows, the equivalent is 1.21.

Conversions are shown in the table that follows:

Approach	Movement	Volume (veh/h)	Equivalent (T 19-5)	Volume (tvu/h)	Lane Group Vol (tvu/h)	No. of Lanes	Vol/Ln (tvu/h)
NB	L	380	1.00	380	380	1	380
	R	360	1.00	360	360	1	360
SB (First St)	L	20	1.21	24	324	1	324
	R	300	1.00	300			
SB (Church Rd)	L	310	1.00	310	334	1	334
	R	20	1.21	24			

Step 3: Determine Sum of Critical Lane Volumes

The figure below shows the ring diagram for the signal, with demand volumes in tvu/h/ln included as appropriate.



V_{cA} = 380 or 360 $\Rightarrow$ 380 tvu/h

V_{cB} = 324 tvu/h

V_{cC} = 334 tvu/h

V_c = 380 + 324 + 334 = 1,038 tvu/h/ln

Note that in Phases B and C, there is only one ring begin used.

Step 4: Determine Yellow and All-Red Intervals
The *yellow* interval is computed using Equation 19-3. Note that as the approach speed is the same for all three approaches, the *yellow* interval will be the same for all three phases.

$$y = t + \frac{1.47\,S_{85}}{2(a + 32.2G)}$$

$$y_{A,B,C} = 1.0 + \frac{1.47(30+5)}{2(10+32.2*0)} = 3.6\ s$$

With few pedestrians, the *all-red* intervals will be computed using Equation 19-4. The intersection diagram, however, does not make it clear what street widths (*w*) vehicles would be crossing during the *all-red* intervals. Obviously, some estimate must be made. In any event, the highest possible width would be the width of Main Street, which is 48 ft. In the absence of exact measurements, this is the value that would be used. Again, since the same street width and speed will be used for all approaches, the *all-red* interval for all phases will be the same.

$$ar = \frac{w + L}{1.47\,S_{15}}$$

$$ar_{A,B,C} = \frac{48+20}{1.47*(30-5)} = 1.9\,s$$

Step 5: Determine the Total Lost Time in the Cycle
The lost time will be the sum of the *yellow* and *all-red* intervals, as the standard default values for ℓ_1 and *e* are being used. Therefore:

$$L = 3*(3.6+1.9) = 16.5\ s$$

Step 6: Determine the Desirable Cycle Length
The desirable cycle length is given by Equation 19-11:

$$C_{des} = \frac{L}{1 - \left[\dfrac{V_c}{1700\,PHF\,(v/c)}\right]}$$

$$C_{des} = \frac{16.5}{1 - \left[\dfrac{1038}{1700*0.85*0.90}\right]} = \frac{16.5}{1-0.798} = 81.7\ \ say\ 85\ s$$

Step 7: Split the Green Time
Green times are split using Equation 19-13. The total green time to be allocated in the 85-sec cycle is 85.0-16.5 = 68.5 s. Then:

$$g_i = g_{TOT}\left(V_{ci}/V_c\right)$$
$$g_A = 68.5\left(380/1038\right) = 25.0s$$
$$g_B = 68.5\left(324/1038\right) = 20.5\,s$$
$$g_C = 68.5\left(334/1038\right) = 22.0\,s$$

Note that 0.1 was arbitrarily deducted from Phase A to insure that the total of allocated green time is 68.5 s. This discrepancy occurred due to the rounding of values to the nearest 0.1 s.

Because the usual default values are in use, actual green times are the same as effective green times.

Step 8: Pedestrian Safety
With a low pedestrian volume, Table 19-6 suggests that a minimum pedestrian WALK interval would be 4.0 s (*PW*). Pedestrians are permitted to cross one street during Phases B and C. The width being crossed in both cases is 24 ft at a walking speed of 3.5 s. Thus, the pedestrian clearance interval for *both* Phases B and C is 24/3.5 = 6.9 s. The minimum time for pedestrians is, therefore:

$$G_p = PW_{\min} + PC = 4.0 + 6.9 = 10.9\ s$$

As the *green* intervals for Phases B and C are both well in excess of what is needed, the signal is safe for pedestrians no matter what pedestrian safety policy is in use.

Note that pedestrians still have a difficult time at this intersection, as they can cross Church Road during Phase B and First Street during Phase C. To cross fully across the intersection, therefore, involves making half the trip in each phase. Special pedestrian signing should be used to alert pedestrians that they cannot cross both streets during one phase, and pedestrian signals should definitely be employed.

Problem 19-7
This intersection is in an urban area, and presents a case of a one-street intersection with a two-way street. There is one opposed LT (the EB LT), which is not insignificant, but there are no exclusive turning lanes provided, which would make providing protection somewhat difficult if it is needed.

Step 1: Develop a Phase Plan

The phase plan will depend entirely on whether the EB LT requires protection or not. There is no LT accident data given, so there are only three criteria that can be checked:

V_{LT} = 80 veh/h < 200 veh/h

xprod = 80*700/2 = 28,000 < 50,000

S_{85} = 30 + 5 = 35 mi/h < 45 mi/h (Table 19-1)

It appears that no LT protection will be needed for this turn, although the moderately high LT volume may still cause a problem in the timing. A simple two-phase signal will be used for this intersection.

Step 2: Convert Volumes to tvu's

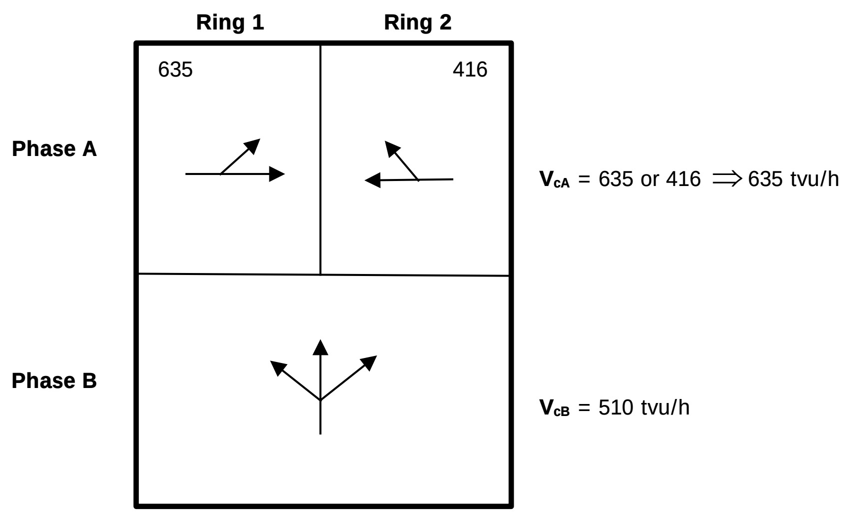
Conversions are shown in the table that follows. Note that left turns from the one-way street are treated as if they were right turns, with pedestrian interference.

Approach	Movement	Volume (veh/h)	Equivalent (T 16-4/5)	Volume (tvu/h)	Lane Group Vol (tvu/h)	No. of Lanes	Vol/Lane (tvu/h)
EB	L	80	*6.50*	520	1270	2	635
	T	750	1.00	750			
WB	T	700	1.00	700	832	2	416
	R	100	1.32	132			
NB	L	100	1.32	132	1530	3	510
	T	1200	1.00	1200			
	R	150	1.32	198			

Italics indicates a value interpolated in Table 19-4.

Step 3: Determine the Sum of Critical Lane Volumes

A ring diagram depicting the phasing and critical lane volumes moving in each is shown in the figure that follows.



Step 4: Determine Yellow and All-Red Intervals

The length of the *yellow* intervals are computed using Equation 19-3. As the approach speed is the same for all approaches, the *yellow* time will be the same for both phases:

$$y = t + \frac{1.47\,S_{85}}{2(a + 32.2G)}$$

$$y_{A,B} = 1.0 + \frac{1.47*(30+5)}{2(10+32.2*0)} = 3.6\ s$$

Because there are a moderate number of pedestrians present, *all-red* times will be determined using Equation 19-6:

$$ar = \frac{P+L}{1.47\,S_{15}}$$

$$ar_A = \frac{(36+10+2)+18}{1.47*(30-5)} = 1.8\ s$$

$$ar_B = \frac{(48+10+2)+18}{1.47*(30-5)} = 2.1\ s$$

Step 5: Determine the Lost Time Per Cycle

Because ℓ_1 and *e* are both 2.0 s, the lost time per cycle is equal to the sum of the *yellow* and *all-red* intervals, or:

$$L = (3.6+1.8) + (3.6+2.1) = 11.1\ s$$

Step 6: Determine the Desirable Cycle Length

The desirable cycle length is computed using Equation 19-11:

$$C_{des} = \frac{L}{1 - \left[\dfrac{V_c}{1700\,PHF\,(v.c)}\right]}$$

$$C_{des} = \frac{11.1}{1 - \left[\dfrac{1145}{1700 * 0.95 * 0.90}\right]} = \frac{11.1}{1 - 0.788} = 52.4\ s\ \ say\ 55\,s$$

Some agencies might prefer to use a 60-sec cycle. The solution will proceed using a 55-sec cycle length.

Step 7: Split the Green Times

The green time will be allocated using Equation 19-13. The total amount of effective green time in the cycle to be allocated is 55.0-11.1 = 43.9 s. Then:

$$g_i = g_{TOT}\left(V_{ci}/V_c\right)$$

$$g_A = 43.9\left(635/1145\right) = 24.3\ s$$

$$g_B = 43.9\left(510/1145\right) = 19.6\ s$$

Because standard defaults are in use, the actual green times are equal to the effective green times.

Step 8: Check Pedestrian Requirements

If moderate pedestrian volumes are equated to “typical,” Table 19-6 suggests that the minimum pedestrian WALK time (for a 55-sec cycle) would be 7.0 s.

The required pedestrian clearance times are computed using Equation 19-16:

$$PC = L/S_p$$

$$PC_A = 36/3.5 = 10.3\ s$$

$$PC_B = 48/3.5 = 13.7\ s$$

Therefore:

$$G_{pA} = 7.0 + 10.3 = 17.3\ s$$

$$G_{pB} = 7.0 + 13.7 = 20.7\ s$$

Phase A will work under any pedestrian safety policy, as the actual green is larger than the minimum pedestrian requirement. Phase B, however, would not work under Option 3, which requires that pedestrians be out of the crosswalk by the end of the vehicular green. The actual green (19.6 s) is 1.1 s less than the minimum pedestrian time required (20.7 s). Under Option 2, the *yellow* interval of 3.6 s could also be used, which would then satisfy the requirement (19.6+3.6 = 23.6 s > 20.7 s). Option 1, which also adds the *all-red* interval would also be acceptable. If Option 3 is required by local policy, the cycle length would have to be increased by 20.7/19.6 = 1.06. A new cycle length of 55*1.06 = 58.3 s, which would be rounded to a 60-sec cycle, would have to be employed. Green times would be re-allocated using Equation 19-13.

Problem 19-8

This intersection is clearly a major one dealing with high volumes of vehicles, moderate pedestrian activity, high turning activity, and high approach speeds.

Step 1: Develop a Phase Plan

Phasing, as always, depends upon whether or not left turns need to be protected. In this case, one criteria alone – an 85th percentile speed of 45 + 5 = 50 mi/h – essentially means that all left turns will have to be protected (Table 19-1). Many other criteria for protection are also exceeded. Suffice it to say that we will be dealing with a 4-phase signal with protected turns for both arteries.

In the N-S street, the LT volumes are nearly equal, and an exclusive LT phase will be provided. In the E-W street, however, LT volumes are very different, and a phasing that allows for different lengths in the protected phases for EB and WB left turns should be the approach. As it is the more standard approach, the solution will use a NEMA phasing with an exclusive LT phase followed by a leading green for the EB (larger) direction.

Note that the very large RT movement from the WB approach may be ignored in the signalization. This is because it has an exclusive lane that does not require merging into the NB departure lanes. The WB RT may move at all times. Therefore, for the sign timing, this volume will be set at “0 veh/h.”

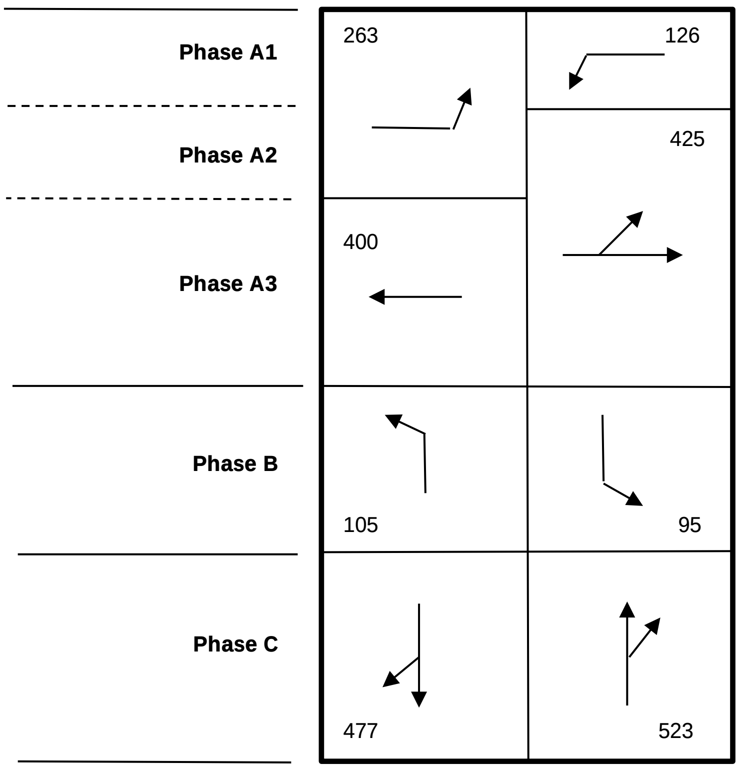
Step 2: Convert Volumes to tvu’s

Tables 19-4 and 16-5 are used to determine equivalents. All of the LTs are protected, and therefore have an equivalent of 1.05. All of the RTs face moderate pedestrian conflicts, and their equivalent is 1.32. Conversions are shown in the table that follows.

Approach	Movement	Volume (veh/h)	Equivalent (T 19-4/5)	Volume (tvu/h)	Lane Group Vol (tvu/h)	No. of Lanes	Volume/Ln (tvu/h)
EB	L	500	1.05	525	525	2	263
	T	1600	1.00	1600	1699	4	425
	R	75	1.32	99			
WB	L	120	1.05	126	126	1	126
	T	1200	1.00	1200	1200	4	400
	R	0	1.32	0			
NB	L	100	1.05	105	105	1	105
	T	1000	1.00	1000	1046	2	523
	R	35	1.32	46			
SB	L	90	1.05	95	95	1	95
	T	900	1.00	900	953	2	477
	R	40	1.32	53			

Step 3: Determine the Sum of Critical Lane Volumes

A ring diagram for the proposed signal timing is shown below, with the critical volumes included as appropriate.



V_{cA} = 263 + 400 = 663 tvu/h

or

V_{cA} = 126 + 425 = 551 tvu/h

V_{cA} = 663 tvu/h

V_{cB} = 105 or 95 $\Rightarrow$ 105 tvu/h

V_{cC} = 523 tvu/h

V_c = 663 + 105 + 523 = 1291 tvu/h

Note that this is a *4-phase* signal plan, as the critical path goes through four separate phases. The cycle will, therefore, include four sets of *yellow* and *all-red* intervals.

Step 4: Determine Yellow and All-Red Intervals

The length of the *yellow* interval is given by Equation 19-3. As the approach speeds are 45 mi/h (average) on all approaches, all phases will have the same *yellow* time.

$$y = t + \frac{1.47\,S_{85}}{2(a + 32.2G)}$$

$$y_{ALL} = 1.0 + \frac{1.47*(45+5)}{2(10+32.2*0)} = 4.6\,s$$

The *all-red* interval is given by Equation 19-5 where moderate pedestrian volumes are in place. Note that turning vehicles are assumed to traverse the same distance as through vehicles, for simplicity.

$$ar = \frac{P+L}{1.47\,S_{15}}$$

$$ar_{A1/A2/A3} = \frac{(72+10+2)+18}{1.47*(45-5)} = 1.7\ s$$

$$ar_{B,C} = \frac{(120+10+2)+18}{1.47*(45-5)} = 2.6\ s$$

Step 5: Determine the Sum of the Lost Times

Because the standard default values for ℓ_1 and *e* are in use, the lost time will be equal to the sum of the *yellow* and *all-red* intervals. Note that in this 4-phase signals, the *yellow* and *all-red* for Phases A1, A2, and A3 occur *twice*. The *yellow* and *all-red* times for Phases B and C occur once for *each phase*. Therefore:

$$L = (4.6+1.7)+(4.6+1.7)+(4.6+2.6)+(4.6+2.6) = 27.0\ s$$

Step 6: Determine a Desirable Cycle Length

The desirable cycle length is determined using Equation 16-11:

$$C_{des} = \frac{L}{1-\left[\dfrac{V_c}{1700\,PHF\ (v/c)}\right]}$$

$$C_{des} = \frac{27.0}{1-\left[\dfrac{1291}{1700*0.95*0.95}\right]} = \frac{27.0}{1-0.841} = 169.9\ s\ \ say\ 170\,s$$

This is an unusually long cycle length, outside the normal range for pretimed signals of 30 – 120 s. Given the parameters of the intersection, unless lanes can be added to some of the approaches, there is almost no way to mitigate it.

Major design alternatives might include banning some left turns, provided there is a reasonable alternative route for these turns to be made, adding lanes – particularly on the N-S artery, and/or considering an overpass for selected through movements.

This solution will proceed with a timing using the large 170-sec cycle length.

Step 7: Splitting the Green

The green is split using Equation 19-13. The total amount of effective green time to be allocated is 170.0 – 27.0 =143.0 s. Then:

$$g_i = g_{TOT}\left(V_{ci}/V_c\right)$$
$$g_{A1+A2} = 143\left(263/1291\right) = 29.1\ s$$
$$g_{A3} = 143\left(400/1291\right) = 44.4\ s$$
$$g_B = 143\left(105/1291\right) = 11.6\ s$$
$$g_C = 143\left(523/1291\right) = 57.9\ s$$

Note that 0.1 was added to Phase A3 to insure that the total green time allocated added up to 143.0 s.

This does not complete the timing, however. The split between Phases A1 and A2, which occurs *only* on the non-critical path, must still be determined. The total length of Phase A is 29.1+44.4 = 73.5 s. This time is now split in the ratio of the lane volumes in each of these phases on the non-critical path:

$$g_{A1} = 73.5\left(126/(126+425)\right) = 16.8\ s$$

By definition, the length of Phase A2 is now:

$$g_{A2} = 73.5 - 44.4 - 16.8 = 14.1\ s$$

All actual green times are equal to the effective green times.

Step 8: Check Pedestrian Safety

Note that in this signal plan, pedestrians will cross the N-S street during Phase A3, and will cross the E-W street during Phase C. From Table 19-6, assuming that moderate pedestrian activity equates to "typical," the recommended minimum pedestrian WALK interval (*PW*) is 7 – 10 s. Given the length of the signal cycle, a value of 10 s adopted.

The pedestrian clearance (*PC*) times are computed using Equation 19-16:

$$PC = L/S_p$$
$$PC_{A3} = 72/4.0 = 18.0\ s$$
$$PC_C = 120/4.0 = 30.0\,s$$

The minimum green times for pedestrians are computed using Equation 19-15:

$$g_{pi} = PW_{\min i} + PC_i$$
$$G_{pA3} = 10.0 + 18.0 = 28.0\, s$$
$$G_{pC} = 10.0 + 30.0 = 40.0\ s$$

As the actual vehicular green times for Phases A3 and C exceed these minima (by quite a bit), the intersection is safe for pedestrians.

Pedestrians, however, are always a problem at such intersections as this – with high speeds, lots of turns, and very wide streets to cross. As there is plenty of green time, the most cautious policy for pedestrians – Option 3 – should be adopted. Pedestrian signals are virtually a must, and design features such as raised crosswalks might also be considered. Should pedestrian accidents become a problem in the future, the option of pedestrian overpasses would be seriously considered, either at the intersection, or at nearby mid-block locations.

Solutions to Problems in Chapter 20

Fundamentals of Signal Timing and Design: Actuated Signals

The following default values are used in both actuated signal timing problems:

- Driver reaction time, t = 1.0 s
- Vehicle deceleration rate, a = 10 ft/s^2
- Length of a vehicle = L = 20 ft
- Start-Up Lost Time, ℓ_1 = 2.0 s.
- Encroachment time, e = 2.0 s.
- Level terrain
- Low pedestrian activity at all locations (50 peds/h each cross walk)
- PHF = 0.90
- Target v/c ratio for actuated signals = 0.95
- Lane widths = 12 ft
- Crosswalk widths = 10 ft with a 2-ft setback
- Pedestrian crossing speed = 4.0 ft/s.
- All volumes in veh/h

Problem 20-1

Note that this is a semi-actuated signal.

(a) Detector Placement

The placement of detectors is related to the desired minimum green time for the side street. Note that for a semi-actuated signal, detectors are only located on the side street. The equation for minimum green time, which (in this case) is fixed at 6.0 s, is:

$$G_{\min,side} = \ell_1 + 2.0\ Int\left[\frac{d}{25}\right]$$

$$6.0 = 2.0 + 2.0\ Int\left[\frac{d}{25}\right]$$

$$Int\left[\frac{d}{25}\right] = \frac{6.0-2.0}{2.0} = 2$$

Any value of "d" between **25.1 and 50.0 ft** will yield *Int* (d/25) = 2. The exact placement of the detector would be based upon local conditions, including driveway and parking place locations.

(b) Passage Time

The recommended passage time for point detectors is the minimum allowable headway, which is **3.0 s.**

(c) Yellow and All-Red Times

Note that *all* semi-actuated signals are two-phase signals. We will assign Phase A to the minor street (N-S) and Phase B to the major street (E-W). Then:

$$y = t + \frac{1.47 * S_{85}}{2(a + 32.2G)}$$

$$y_A = 1.0 + \frac{1.47*(25+5)}{2(10+32.2*0)} = 1.0 + 2.2 = 3.2\ s$$

$$y_B = 1.0 + \frac{1.47*(45+5)}{2(10+32.2*0)} = 1.0 + 3.7 = 4.7\ s$$

Given low levels of pedestrian activity, the all-red time will be based on clearing a vehicle through a distance of w + L, where w is the width of the street being crossed. Then:

$$ar = \frac{w + L}{1.47 * S_{15}}$$

$$ar_A = \frac{48 + 20}{1.47*(25-5)} = 2.3\ s$$

$$ar_B = \frac{24 + 20}{1.47*(45-5)} = 0.7\ s$$

(d) Maximum Side-Street Green, Minimum Main Street Green

For a semi-actuated signal, the detector placement determines the minimum side-street green. As there are no detectors on the main street, the minimum main street green is set through signal timing. A critical cycle consists of the maximum side-street green, the minimum main street green, and lost times. To obtain the maximum side-street green and minimum main street green, the signal is treated as if it were pre-timed. First, however, all volumes must be converted to through car equivalents, as shown in the table below.

Movement	**Volume (veh/h)**	**Equivalent (Tables 19-4/5)**	**Volume (tvu/h)**	**No. of Lanes**	**Volume (tvu/h/ln)**
EB LT	5	6.50*	33	2	(33+650+12)/2= 348
EB TH	650	1.00	650		
EB RT	10	1.21	12		
WB LT	5	7.25*	36	2	(36+700+13)/2= **375**
WB TH	700	1.00	700		
WB RT	11	1.21	13		
NB LT	12	2.50	30	1	30+150+12= 192
NB TH	150	1.00	150		
NB RT	10	1.21	12		
SB LT	15	2.15*	32	1	32+200+18= **250**
SB TH	200	1.00	200		
SB RT	15	1.21	18		

*Interpolation required.

Because this is a simple 2-phase signal, the critical volumes are 375 tvu/h/ln (WB) and 250 tvu/h/ln (SB). The sum of critical lane volumes is 375+250 = **625 tvu/h/ln.**

Because the start-up lost time and encroachment times are equal (2.0 s each), the lost time per cycle is equal to the sum of the yellow and all-red intervals:

$$L = (3.2 + 2.3) + (4.7 + 0.7) = 10.9\ s$$

The initial cycle length used to allocate green will be:

$$C = \frac{L}{1 - \left[\dfrac{V_c}{1700 * PHF * (v/c)}\right]}$$

$$C = \frac{10.9}{1 - \left[\dfrac{625}{1700 * 0.90 * 0.95}\right]} = \frac{10.9}{1 - 0.430} = 19.1\ s$$

For an actuated signal, this value is used directly without rounding. Unfortunately, this is a very small cycle length. When the lost time of 10.9 s is accounted for, only 19.1 – 10.9 = 8.2 s is left to allocate to the two green times. While we know the results will not be workable, we will continue the solution for illustrative purposes:

$$g_{A,MAX} = 8.2\left({}^{250}\!/\!_{625}\right) = 3.3\ s * 1.5 = 5.0\ s$$
$$g_{B,MIN} = 8.2\left({}^{375}\!/\!_{625}\right) = 4.9\ s * 1.5 = 7.4\ s$$

Note that as an actuated signal, both times are multiplied by a factor of 1.5 to insure that there is flex in the cycle even during periods of peak loading.

Clearly, this is not adequate, as the maximum for Phase A is 5.0 s, with a minimum of 6.0 s! Obviously, the maximum must be more than the minimum. The minimum will allow for 2 vehicles to proceed through the minor street green. A reasonable maximum must now be assumed that allows some greater number of vehicles to proceed. Given the low volumes on the side street, a maximum allowing 3 or 4 vehicles to proceed would be expected. Working with 2 additional vehicles (for a maximum of 4), 2 seconds of green must be added for each vehicles, making a reasonable **$g_{A,max}$ = 6.0+2*2 = 10.0 s.**

The minimum green for Phase B must now be increased to keep the proportioning of green equal to the original ratio, or:

$$\frac{7.4}{5.0}=\frac{g_{B,\min}}{10.0}$$

$$g_{B,\min}=\frac{7.4*10.0}{5.0}=14.8\ s$$

Therefore, the following timing would be implemented. Note that as the start-up lost time and encroachment time are equal, the actual green times equal the effective green times:

Phase	Minimum Green (G_{min})	Maximum Green (G_{max})	Yellow	All-Red
A (Side Street, N-S)	6.0 s	10.0 s	3.2 s	2.3 s
B (Main Street, E-W)	14.8 s	NA	4.7 s	0.7 s

(e) Critical Cycle Length

The critical cycle length for a semi-actuated signal is the sum of maximum green time for the side street, the minimum green time for the main street, plus all yellow and all red times. For this signal:

$$C_{cr}=10.0+14.8+3.2+4.7+2.3+0.7=35.7\ s$$

(f) Pedestrians

Pedestrians Crossing the Minor Street

Pedestrians crossing the minor (or side) street will do so during Phase B, during which they have a *minimum* of 14.8 s of green time. This must be compared to the minimum pedestrian crossing time required:

$$G_{pi}=PW_{\min i}+PC_i=PW_{\min i}+\left(L/S_p\right)$$

From text Table 19-6, the minimum *PW* interval for low (or negligible) pedestrian activity is 4.0 s. Then:

$$G_{pA}=4.0+\left(24/4\right)=10.0\ s$$

Pedestrians crossing the minor (or side) street are safely accommodated by the signal timing.

Pedestrians Crossing the Major Street

Pedestrians crossing the major street do so during Phase A, during which only 6 s of green time is assured. Further, if there are no vehicles present on the side street, Phase A will never be implemented. The minimum pedestrian green time for Phase A is:

$$G_{pB} = 4.0 + \left(48/4\right) = 16.0\ s$$

Obviously, pedestrians cannot safely cross the street during the minimum green time for Phase A. A pedestrian push-button is required in any case, so that a pedestrian can get green phase when there are no vehicles present. Now, pedestrian signals will have to be added (for pedestrians crossing the major street). *Unless* the pedestrian button is pushed, they will show "DON'T WALK" at all other times. When the pedestrian button is pushed, the pedestrian crossing time of 16.0 s will have to be provided. Local policy now enters the picture: are pedestrians allowed to be crossing during yellow and all-red intervals (Option 1)? Depending upon local policy, the 16.0 s can be made up of green only (Option 3), green plus yellow (Option 2), or green plus yellow plus all-red (Option 1). In all cases, the WALK interval will be 4.0 s. The remaining time will be the pedestrian clearance, or Flashing DON'T WALK. The table below shows how the signal would react when the pedestrian push-button is activated (on the NEXT green phase), depending upon local policy:

Policy	**G (s)**	**y (s)**	**ar (s)**	**WALK (s)**	**Flashing DON'T WALK (s)**
Green Only	16.0	3.2	2.3	4.0	48/4 = 12.0
Green+Yellow	16.0-3.2=12.8	3.2	2.3	4.0	12.0
Green+Yellow+ All Red	16.0-3.2-2.3= 10.5 s	3.2	2.3	4.0	12.0

As a semi-actuated signal, the dual entry and simultaneous force off switches will both be "on" for both phases. There is no recall on Phase A, but a maximum recall would be in effect for Phase B.

Problem 20-2

Note that this is a full-actuated signal.

(a) Phase Plan

The selection of an appropriate phase plan depends upon whether or not any left-turn movements require protection. The three criteria for initial consideration of left-turn protection are considered in the table below.

Left Turn Movement	Is $v_{LT} \geq 200$ veh/h?	Is xprod ≥ 50,000 ?	Criteria of Table 19-1
EB	No	7*(850/2)=2,975 No	No
WB	No	5*(800/2)= 2,000 No	No
NB	No	10*(750/2)=3,750 No	No
SB	No	10*(700/2)=3,500 No	No

It is clear from the criteria that all left turns may be handled on a permitted basis. Further, none of the LT volumes are significant enough to warrant considering protection at a lower threshold and no LT lanes exist. Therefore, this will be a simple two-phase signal. Phase A will be for the E-W street and Phase B for the N-S street.

(b) Minimum Green Times

As the point detector locations are fixed, the minimum green times are based upon the distance between the detector and the STOP line:

$$G_{\min} = \ell_1 + 2.0\ Int\left[\frac{d}{25}\right]$$

$$G_{A,\min} = 2.0 + 2.0\ Int\left[\frac{60}{25}\right] = 2.0 + (2.0*3) = 8.0\ s$$

$$G_{B,\min} = 2.0 + 2.0\ Int\left[\frac{30}{25}\right] = 2.0 + (2.0*2) = 6.0\ s$$

(c) Passage Time

For point detectors, the passage time, PT, is equal to the minimum allowable headway (MAH), which is a standard **3.0 s.**

(d) Yellow and All-Red Times

Because in this case, all average approach speeds are equal, and the width of both streets are the same (48 ft), the yellow and all-red intervals for both phases will be the same. Note that the 85th percentile speed is estimated as 5 mi/h more than the average approach speed, and that the 15th percentile speed is estimated as 5 mi/h less than the average approach speed. Because there is low pedestrian activity, the all-red interval is timed to allow vehicles to clear a distance of w + L ft. Then:

$$y = t + \frac{1.47\,S_{85}}{2(a + 32.2G)}$$

$$y_{A,B} = 1.0 + \frac{1.47(45+5)}{2(10+32.2^*)} = 1.0 + 3.7 = 4.7\ s$$

$$ar = \frac{w+L}{1.47\,S_{15}}$$

$$ar_{A,B} = \frac{48+20}{1.47(45-5)} = 1.2\ s$$

(e) Maximum Green Times

Maximum green times are found by considering the signal timing as if it were pretimed – then multiplying the results by 1.5. To determine a cycle length for these computations, all demand volumes must be converted to tvu/h/ln, as shown in the table below:

Movement	**Volume (veh/h)**	**Equivalent (Tables 16-5 and 16-6)**	**Volume (tvu/h)**	**No. of Lanes**	**Volume (tvu/h/ln)**
EB LT	7	9.25*	65	2	(65+800+12)/2= 439
EB TH	800	1.00	800		
EB RT	10	1.21	12		
WB LT	5	8.0	40	2	(40+850+18)/2= **454**
WB TH	850	1.00	850		
WB RT	15	1.21	18		
NB LT	10	7.25*	73	2	(73+700+18)/2= 396
NB TH	700	1.00	700		
NB RT	15	1.21	18		
SB LT	10	6.50*	65	2	(65+750+18)/2= **417**
SB TH	750	1.00	750		
SB RT	15	1.21	18		

*Interpolation required.

The critical lane volumes in this case are clear. For a two phase signal, the largest E-W volume and the largest N-S volume are critical, in this case WB and SB. The sum of critical lane volumes is 454+417 = **871 tvu/h/ln**.

Because start-up lost time and encroachment time are equal (2.0 s each), the lost time per cycle is the sum of the yellow and all-red intervals, or:

$$L = (4.7+1.2)+(4.7+1.2) = 11.8\ s$$

Then:

$$C=\frac{L}{1-\left[\frac{L}{1700*PHF*(v/c)}\right]}$$

$$C=\frac{11.8}{1-\left[\frac{871}{1700*0.90*0.95}\right]}=\frac{11.8}{1-0.599}=29.4\ s$$

The amount of effective green time to allocate 29.4-11.8 = 17.6 s, which is allocated in the ratio of critical lane volumes (then multiplied by 1.5):

$$g_{A,\max}=17.6\left(\tfrac{454}{871}\right)=9.2\ s\ x\ 1.5=13.8\ s$$

$$g_{B,\max}=17.6\left(\tfrac{417}{871}\right)=8.4\ s\ x\ 1.5=12.8\ s$$

Because the start-up lost time and encroachment time are equal, the actual maximum green times and the effective maximum green times are the same.

Both maximum green times are in excess of the minimum green times (8.0 s for Phase A, and 6.0 s for Phase B), although not greatly. These are reasonable values unless other related conditions suggest a need for a higher maximum greens.

(f) Critical Cycle Length

The critical cycle length for a full-actuated signal is the sum of the maximum green times plus all yellow and all-red intervals, or:

$$C_{cr}=13.8+12.8+(4.7+1.2)+(4.7+1.2)=37.4s$$

(g) Pedestrians

In this case, because both streets are of equal width, the minimum pedestrian crossing time is the same for both phases:

$$G_{pi}=PW_{\min i}+PC_i=PW_{\min i}+\left(\tfrac{L}{S_p}\right)$$

$$G_{pA,B}=4.0+\left(\tfrac{48}{4.0}\right)=16.0\ s$$

Whether or not this is safe depends upon the pedestrian policy in place. Obviously, if pedestrians are only permitted in the crosswalk during green, neither phase provides sufficient time.

If pedestrians may complete crossing during *yellow*, the minimum crossing time provided is the minimum green plus *yellow*. For Phase A, this is 8.0+4.7 = 12.7 s; for Phase B, it is 6.0+4.7 = 10.7 s, neither of which is sufficient. If the *all red* is also added, the crossing time becomes 8.0+4.7+1.2 = 13.9 s for Phase A, and 6.0+4.7+1.2 = 11.9 s, neither of which is sufficient.

Thus, none of the policies would provide for safe crossings, and pedestrian push-buttons and signals would be required on both streets. The following table indicates the timing that would result from pedestrian actuation.

Policy	**G (s)**	**y (s)**	**ar (s)**	**WALK (s)**	**Flashing DON'T WALK (s)**
Green Only	16.0	4.7	1.2	4.0	48/4 = 12.0
Green+Yellow	16.0-4.7=11.3	4.7	1.2	4.0	12.0
Green+Yellow+ All Red	16.0-4.7-1.2 =10.0 s	4.7	1.2	3.5	12.0

With a full-actuated two-phase signal, the dual entry and simultaneous force off features would be "on" in both phases. A minimum or soft recall could be placed on the street considered to be the major street.

Problem 20-3

(a) Signal Phasing
This is a case in which none of the left turns meet any of the criteria for left-turn protection in the strict sense. However, the EB and WB LTs are not insignificant, and the cross-products, while not exceeding 50,000, are high. Given that this is an actuated signal, and that EB and WB LT lanes are provided, it is likely that provision of protected turns for these movements would be implemented.

Because the opposing EB and WB left turns have quite different volumes, one would normally opt for an overlapping phase plan, usually a NEMA-type phasing. For an actuated signal, this is provided for using the "Dual Entry" and "Simultaneous Force-Off" settings. The timing, however, is approached assuming an exclusive left-turn phase for EB and WB left-turns, as this would yield the longest critical cycle length.

(b) Minimum Green Times
All detectors are 40-ft area detectors. Thus, the minimum green time will vary according to the equation:

$$g_{\min} = \ell_1 + 2n$$

where ℓ_1 is 2.0 s, and n may vary from 1 to 2 vehicles. Thus, the minimum green time may vary between 4 s and 6 s.

(c) Passage Time: Passage time for area detectors is given by Equation 20-5, with a maximum allowable headway of 3 s:

$$PT = MAH - \frac{L_v + L_d}{1.47\, S_a} = 3.0 - \left(\frac{20+40}{1.47*45}\right) = 3.0 - 0.9 = 2.1\ s$$

(d) Yellow and All-Red Times: Because there are three phases in the signal, there will be three sets of yellow and all-red times – two associated with the E-W street, and one associated with the N-S street. Because the phasing will ultimately NOT involve three discrete phases, we will label the three phases as EWLT, EWTH/RT, and NS.

(e) Yellow times are computed using Equation 20-6 of the textbook. For low pedestrian flows, w+L will be used in Equation 20-7 to determine *all-red* times:

$$y_{EWLT,EWTH/RT,NS} = 1.0 + \left[\frac{1.47*(40+5)}{2\ (10+32.2*0)}\right] = 1.0 + 3.3\ s = 4.3\ s$$

$$ar_{EWLT.EWTH/RT} = \frac{48+20}{1.47*(40-5)} = 1.3\ s$$

$$ar_{NS} = \frac{60+20}{1.47*(40-5)} = 1.6\ s$$

Because the approach speeds are the same on both streets, all of the yellow intervals (there are three of them) are the same.

(f) Maximum Green Times

Maximum green times are obtained by assuming the signal is pretimed, and then multiplying the final green times by 1.5, to allow for flexibility in even the worst 15 minutes of the peak hour. To do this, the critical path through the signal phasing must be found. First, all demand volumes must be converted to tvu/h/ln.

Through vehicle units for all demand volumes are estimated using Tables 19-4 and 19-5 of the textbook:

Approach	Mvt	Vol (veh/h)	Equiv	Vol (tvu/h)	Lane Grp Vol (tvu/h)	Ln Grp Vol (tvu/h/ln)
EB	**L**	180	1.05	189	189	**189**
	TH	400	1.00	400	461	461/2= 231
	R	50	1.21	61		
WB	**L**	80	1.05	84	84	84
	TH	500	1.00	500	536	536/2= **268**
	R	30	1.21	36		
NB	**L**	10	4.80*	48	718	718/2= **349**
	TH	650	1.00	650		
	R	15	1.21	18		
SB	**L**	10	5.75*	58	650	650/2= 325
	TH	580	1.00	580		
	R	10	1.21	12		

*interpolation required

As we are treating the timing as if this were a simple three-phase signal, the sum of the critical lane volumes consists of:

- Maximum of the EBLT or WBLT
- Maximum of the EBTH/RT or WBTH/RT
- Maximum of the NB (all) or SB (all)

Thus:

$$V_c = 189 + 268 + 349 = 806\ tvu/h/\ln$$

The total lost time per cycle is also needed to find the cycle length and allocate green time. Because standard default values of 2.0 s each are used for both ℓ_1 and e, the lost time/cycle is equal to the sum of the three yellow plus all-red times:

L = (4.3+1.3) + (4.3+1.3) + (4.3+1.6) = 17.1 s/cycle

The desirable cycle length may now be computed as:

$$C = \frac{17.1}{1-\left[\dfrac{806}{1700*0.90*0.95}\right]} = \frac{17.1}{1-0.555} = 38.4\ s$$

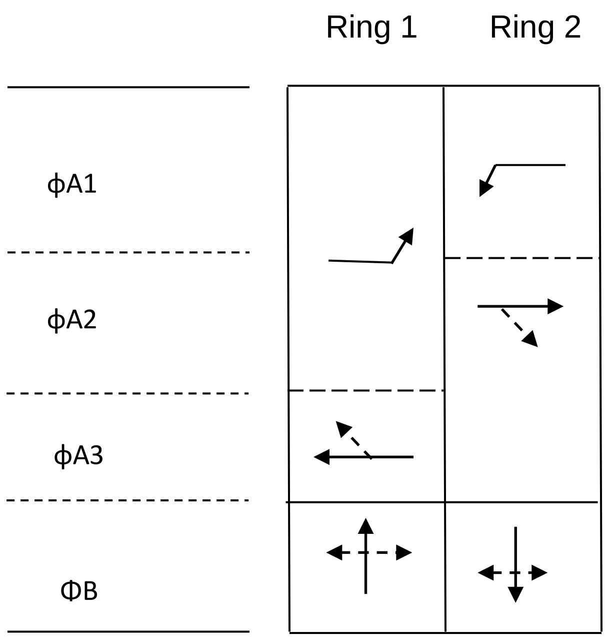
Once again, maximum green times are set by splitting the green within the desirable cycle length, and multiplying the results by 1.5 to assure flexibility even in peak periods:

$$g_{TOT} = C - L = 38.4 - 17.1 = 21.3\,s$$
$$g_{EWLT} = 21.3 * \left(189/806\right) = 5.0\ s\ \ x\ 1.5 = 7.5\ s$$
$$g_{EWTH/RT} = 21.3 * \left(268/806\right) = 7.1\ x\ 1.5 = 10.7\ s$$
$$g_{NS} = 21.3 * \left(349/806\right) = 9.3\ s\ \ x\ 1.5 = 14.0\ s$$

As all of the maximum greens are higher than the minimum greens, this would be an acceptable timing, although the LT phase would have very little flexibility.

(g) Critical Cycle Length
The critical cycle length is the sum of the maximum green times plus lost times, or 7.5+10.7+14.0+17.1 = 49.3 s.

(h) Dual Entry and Simultaneous Force-Off Settings
This issue must be very carefully considered. We wish to allow overlaps to occur, using a NEMA phasing sequence for the E-W street (N-S is a single combined phase). Consider the following ring diagram which shows an expected cycle with more EB left turns than WB left turns:



EB and WB left turns would normally start on Rings 1 and 2 at the same time, but do not have to. When one or both left turns have no demand, one LT could be initiated without the other. The WB TH/RT movements begin when the EB left-turn phase terminated. The EB TH/RT movements begin when the WB left-turn phase is terminated. Clearly, they do not have to start at the same time, but they MUST end at the same time, as the green will be handed off to conflicting N-S movements. The NB and SB movements both begin and end at the same time. Thus, the following Dual Entry and Simultaneous Force-Off settings are necessary:

Movement	Dual Entry Setting	Simultaneous Force-Off Setting
EB LT & WB LT	Off	Off
EBTH/RT & WBTH/RT	Off	On
NB & SB	On	On

A minimum or soft recall could be placed on the through phase for the major street, although this designation is not clear from the volumes and other information presented.

(i) Pedestrians

Obviously, none of the minimum crossing times would be satisfied by the minimum green times, which range between 4 and 6 s. Pedestrians will cross the N-S street during the EBTH/RT & WBTH/RT phase. Pedestrians will cross the E-W street during the NB/SB phase. Then:

$$G_{pi} = PW_{\min i} + PC = PW_{\min i} + \left(L/S_p \right)$$
$$G_{pEWTH} = 4.0 + \left(48/4.0\right) = 16.0\ s$$
$$G_{pNS} = 4.0 + \left(60/4.0\right) = 19.0\ s$$

Pedestrian signals and push-buttons would be used on both streets. The WALK + Flashing DON'T WALK interval would be increased to 16.0 s during the EBTH/RT and WBTH/RT phases and 19.0 s would be implemented during the NB/SB phase, when the pedestrian push-button is activated. Local policy on pedestrian use of yellow and all-red intervals would also be in effect.

Solutions to Problems in Chapter 21

Signal Coordination for Arterials and Networks

Problem 21-1

(a) Offset, with a Moving Platoon upstream:

$$t = \frac{L}{S} = \frac{1000}{40*1.47} = 17.0\ s.$$

(b) Offset, with a Standing Platoon upstream:

$$t = \ell_1 + \frac{L}{S} = 2.0 + 17.0 = 19.0\ s$$

(c) Offset with Queue of 3 Vehicles Downstream:

$$t = \frac{L}{S} - (Qh + \ell_1) = 17.0 - (3*2+2) = 9.0\ s$$

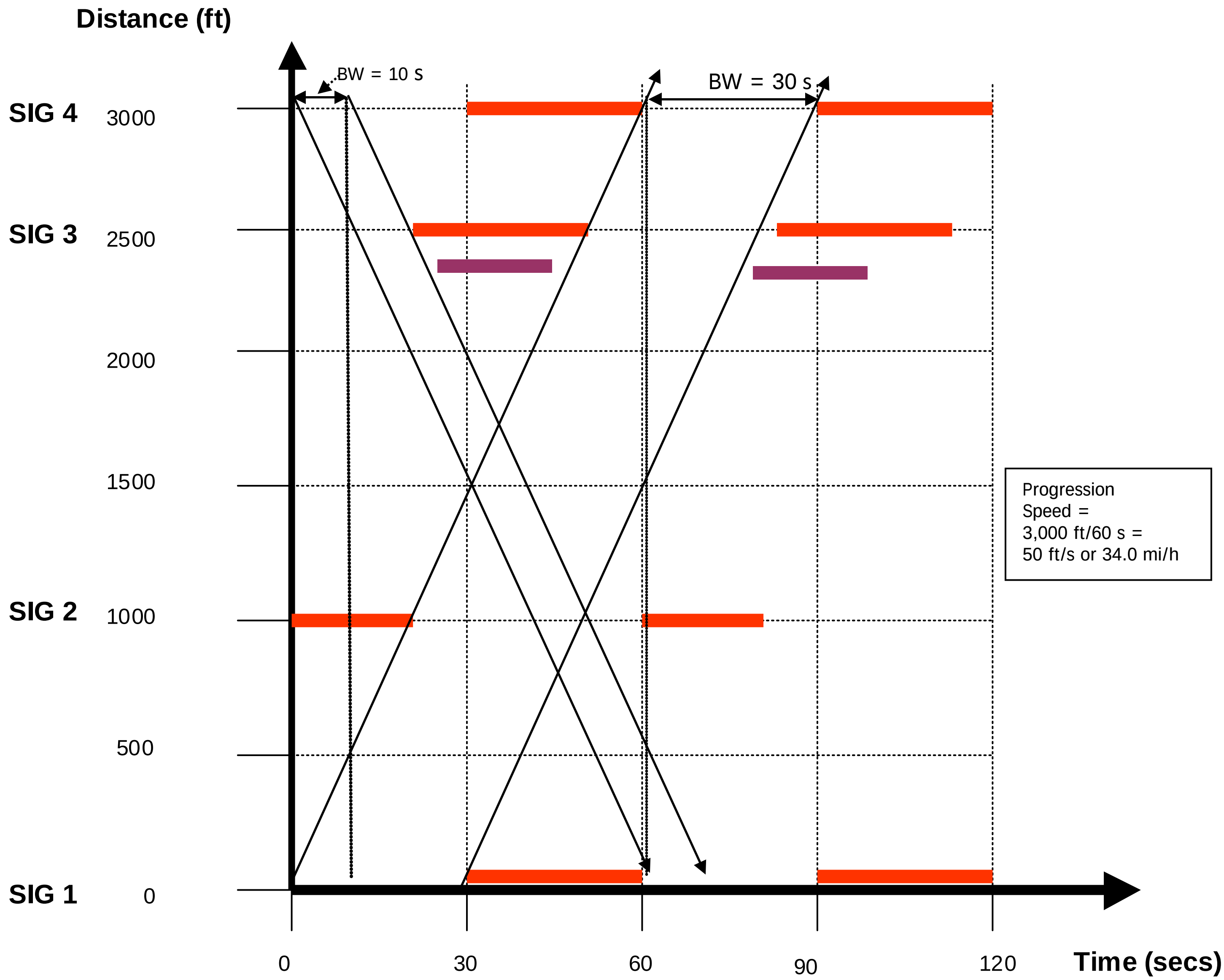
(d) Offset in the opposite direction:

The sum of the offsets in each direction must be equal to an even multiple of the cycle length. In this case, vehicles can make a "round trip" within one cycle. Thus, the offset in the opposite direction is 60.0 – 17.0 = 43.0 s. This would result in platoons arriving most likely during the red interval, and high delays would occur.

(e) Effect of a Poor Speed Estimate:

If the actual desired speed were 45 mi/h, the ideal offset should have been 1000/(45*1.47) = 15.1 s. The lead vehicle of the platoon would, therefore, arrive 17.0-15.1 = 1.9 s "early," i.e., before the green is initiated. This would add 1.9 s of delay to each vehicle in the platoon.

Problem 21-2



Time-Space Diagram for Problem 21-2

(b) NB bandwidth is 30 seconds, NB bandwidth capacity is:

NB bandwidth capacity = 3,600 * 30/(60 * 2) = 900 vphpl

(c) SB bandwidth is 10 seconds

SB bandwidth capacity = 3,600 * 30/(10*2) = 300 vphpl

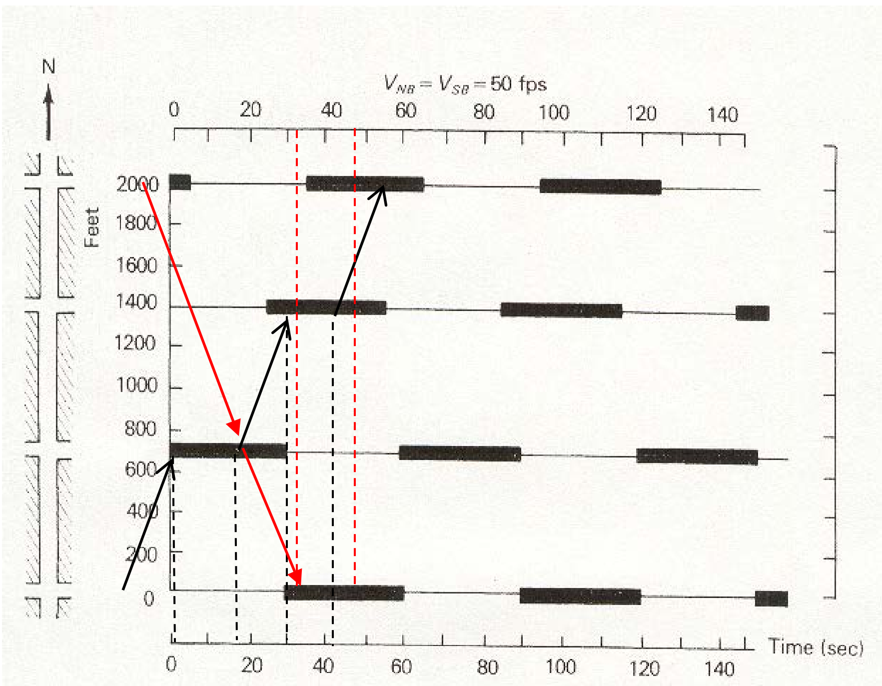
(d) You place the driveway where it will not reduce the bandwidth in either direction.

Problem 20-3

The travel time would be 750 ft / (30*1.47) = 17 s. Thus for an alternate progression the cycle length needed would be 17*2=34 (rounded to 35 s). For a double alternate progression, the cycle length needed would be twice this or 70 sec.

Although the bandwidth is larger with an alternate progression, is a 35 second cycle length practical? Will a 35-sec cycle have adequate capacity for all movements? Thus the engineer must look at the entire system and make a judgement.

Problem 21-4



Time-Space Diagram for Problem 21-4

Northbound Vehicle

At 50 ft/s, the lead NB vehicle will arrive at the first signal (700 ft) in 700/50 = 14 s. It will stop, and will leave when the green begins at 30 s, compiling 16 s of delay plus 2 seconds start-up lost time. It stops again at the second signal (1400 ft) at 32 + 14 = 46 s, and departs on the green at 55 s, compiling another 11 s plus 2 seconds start-up lost time of delay. The vehicle will not have to stop at the third signal. Result: 2 Stops; 29 s of delay.

Southbound Vehicle

At 50 ft/s, the lead SB vehicle does not stop at the third signal nor at the second. It does stop at the first signal, however. Since the green at signal 3 starts at 5 s, and it takes the

SB vehicle 2000/50 = 40 s to get to the first signal, it will stop at 45 s, and leave at the green, which begins at 60 s. Result: 1 stop; 15+2 =17 s of delay.

These answers assume that the lead vehicle is moving when it hits the initial green phase.

Problem 21-5

The network is 2000 feet, at 50 fps it will take 40 seconds to travel the entire arterial.

Northbound:

The intersection at 500 feet limits the first vehicle to get through the system. It takes ten seconds to get to that second intersection, so the first vehicle could not pass through the second intersection, which is red at time=10 seconds. That first vehicle moving through intersection one to not stop at intersection two starts at 10 seconds and will arrive at the last intersection at 50 seconds. The last vehicle can pass the first intersection at 20 seconds, and make it through to the last intersection at 60 seconds, without stopping. **Thus the Northbound bandwidth is 10 seconds, with an efficiency of 10/60 = 16.6%**

Southbound:

The southbound bandwidth is five seconds, with an efficiency of 8.3%

NB capacity = [3600*10*3]/[60*2] = 900 vph.

SB capacity = 450 vph

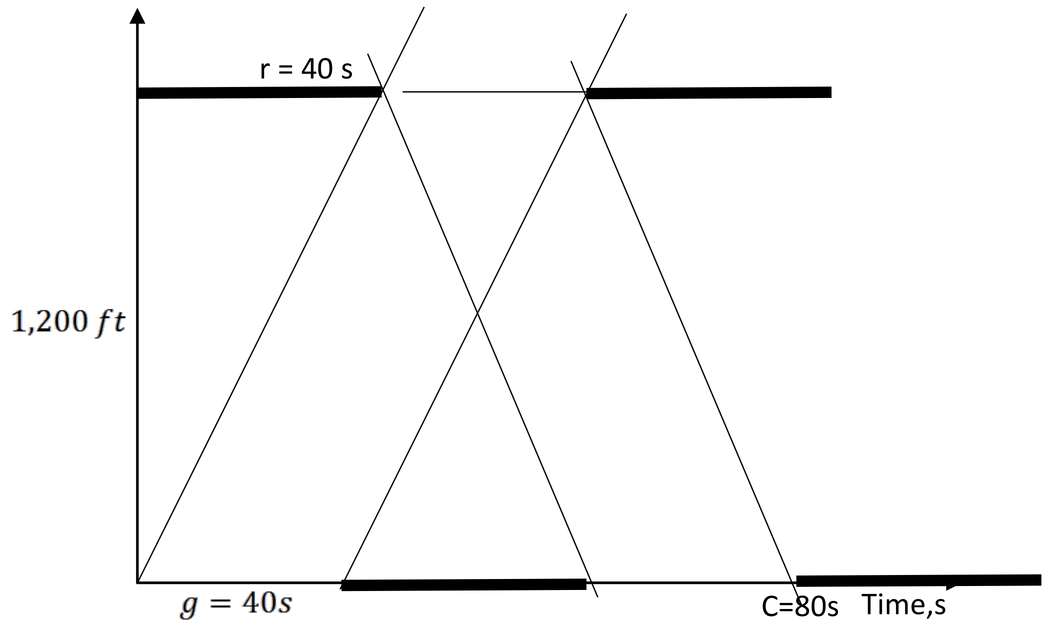
Problem 21-6

(a) First check "natural" spacings.

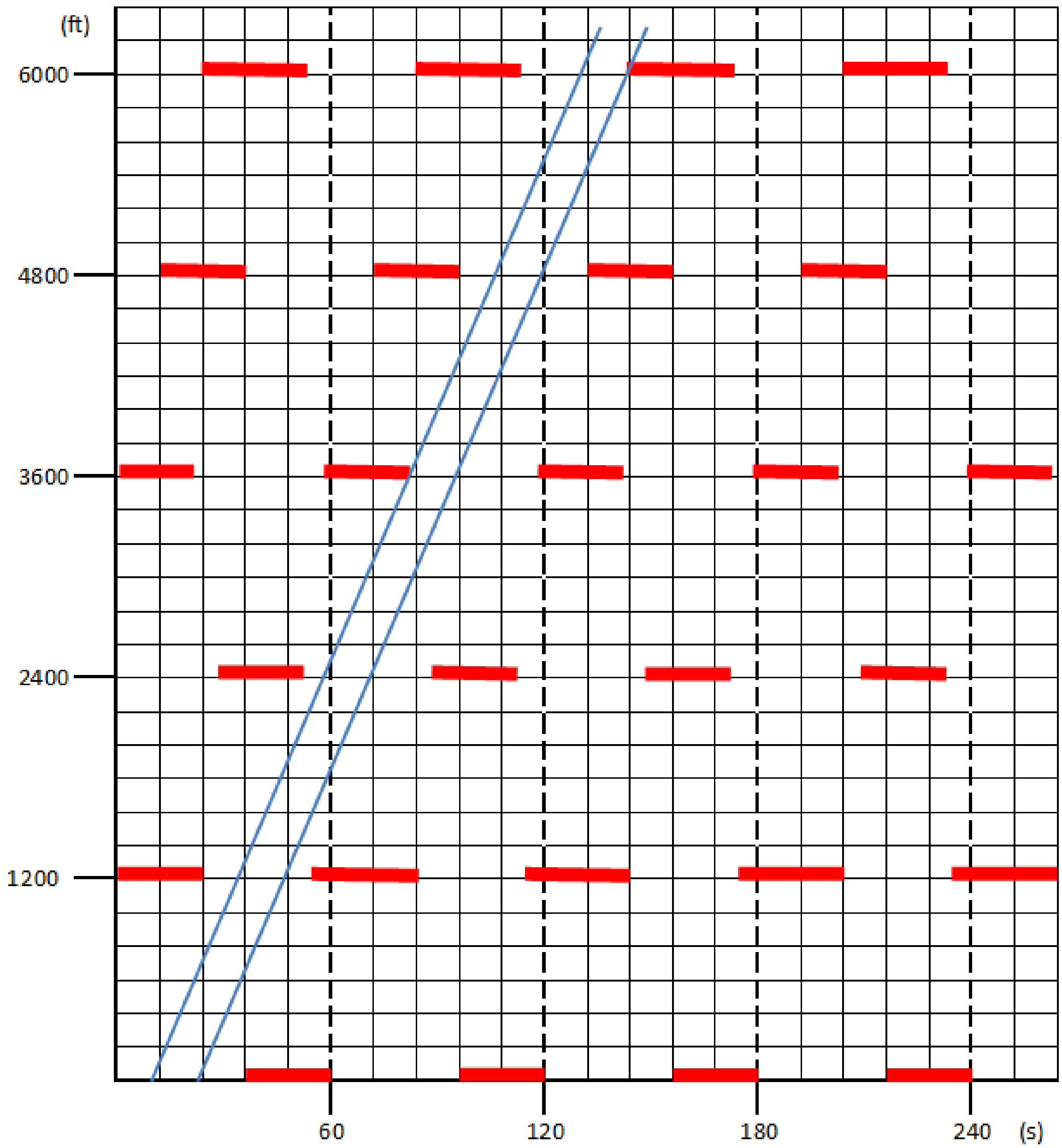
An alternate progression requires L/V = C/2; and C = (2,400/60) * 2 = 80 seconds. Thus use 80 seconds with an alternate progression

(b) Refer to the plot of an alternate system. At the midpoint, first passes the NB platoon, and then the SB platoon. Thus, with heavy flows in both directions, there may never be a significant time without activity for vehicles at the unsignalized intersection to find gaps. Thus these vehicles will experience high delay.

If the intersection could be placed at 600 feet, it could be "hidden" in the shadow of the platoon movements where gaps would be found.


r = 40 s
1,200 ft
$g = 40s$
C=80s
Time,s

Problem 21-7



(b) From the time-space diagram above we can see that the northbound bandwidth is **14 sec**. This gives us an efficiency of:

$$EFF = \frac{14}{60} \cdot 100 = 23.3\ \%$$

The nonstop northbound capacity (assuming 2.0 of saturation headway) is:

$$C_{BW} = \frac{3{,}600 \cdot BW \cdot L}{C \cdot h} = \frac{3{,}600 \cdot 14 \cdot 2}{60 \cdot 2} = 840\ \ veh/h$$

From the time-space diagram we can see that there cannot be nonstop southbound movement. Southbound bandwidth is 0 sec.

Problem 21-8

Start at the intersection of Avenue A and 3rd Street moving northbound, and travel around the network finding starts of green. (See Table A below) Then find the missing offsets that are predetermined because it is a closed network (See Table B.)

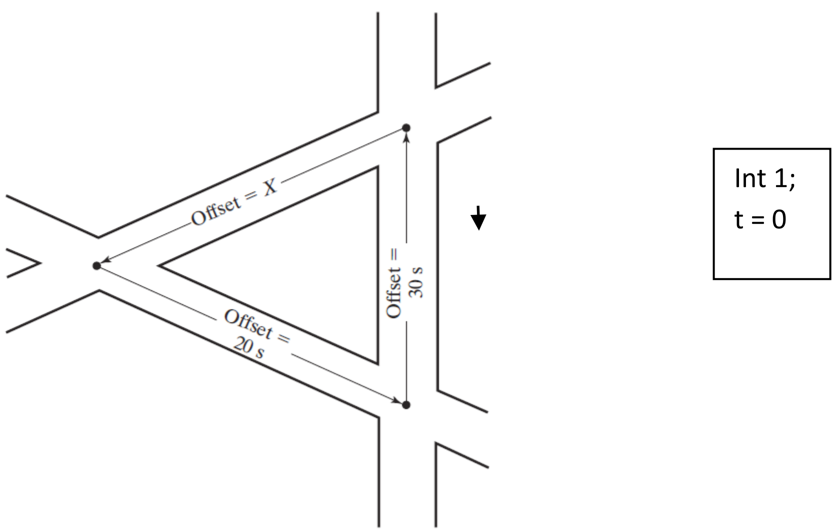
Table A: Start of Green Times

On	At	Start of Green
3rd Street	Avenue J	0
3rd Street	Avenue I	0+10 =10
3rd Street	Avenue H	10+10 = 20
3rd Street	Avenue G	30
3rd Street	Avenue F	40
3rd Street	Avenue E	50
3rd Street	Avenue D	60 = 0
3rd Street	Avenue C	10
3rd Street	Avenue B	20
3rd Street	Avenue A	30
Ave A	3rd Street	30+36 = 66 = 6
Ave A	2nd Street	6+20 = 26
2nd Street	Avenue A	26 + 24 = 50
2nd Street	Avenue B	50+15 = 65 = 5
2nd Street	Avenue C	5+15 = 20
2nd Street	Avenue D	20+15 = 35
2nd Street	Avenue E	35+15 = 50
2nd Street	Avenue F	50+15=65 = 5
2nd Street	Avenue G	5+15 = 20
2nd Street	Avenue H	20+15 = 35
2nd Street	Avenue I	35+15 = 50
2nd Street	Avenue J	50+15=65 = 5
Ave J	2nd Street	5+36 = 41

Table B: Missing Offsets

Intersection	Relative to Intersection	Offset = X
Ave J and 3rd St	Ave J and 2nd St	41 + X + 24 = 0 5 + X = 60 X = 55
Ave I and 2nd St	Ave I and 3rd St	10+36+X+24= 50 X = 40
Ave H and 3rd St	Ave H and 2nd St	35 + X = 20 X = 45
Ave G and 2nd St	Ave G and 3rd St	30 + X = 20 X = 50
Ave F and 3rd St	Ave F and 2nd St	5 + X = 40 X = 35
Ave E and 2nd St	Ave E and 3rd St	50 + X = 50 X = 0
Ave D and 3rd St	Ave D and 2nd St	35 + X = 0 X = 25
Ave C and 2nd St	Ave C and 3rd St	10 + X = 20 X = 10
Ave B and 3rd St	Ave B and 2nd St	5 + X = 20 X = 15

Problem 21-9



Start at Intersection one, t = 0; Intersection 2, t = 0 + 30 = 30
Intersection 2, opposite direction green starts at t = 30 + 40 = 70
Intersection 3, t = 70 + 20 = 90 = 10 s
Intersection 3, opposite direction green starts at t = 10 + 40 = 50
Intersection 1, 50 + X + 40 = 0; 10 + X = 80; X = 70 sec

SOLUTIONS TO PROBLEMS IN CHAPTER 22

CAPACITY AND LEVEL OF SERVICE ANALYSIS: SIGNALIZED INTERSECTIONS

Problem 22-1

The intersection described has three lane groups: a single exclusive left-turn lane, a single exclusive right-turn lane, and two through-only lanes.
The saturation flow rate for a lane group is given by Eqn 22-12:

$$s = s_o\, N f_w\ f_{HVg}\ f_p\ f_{bb}\ f_a\ f_{LU}\ f_{RT}\ f_{LT}\ f_{Rpb}\ f_{Lpb}\, f_{wz}$$

where: s_o = 1,900 pc/hg/ln (default value)

The factors for lane width, heavy vehicles, grade, and area type will be the same for all the three lane groups and are computed as follows:

$$f_w = 1$$
$$f_{HVg} = \frac{100 - 0.78P_{HV} - 0.31G^2}{100} = \frac{100 - 0.78*10 - 0.31*3}{100} = 0.894$$
$$f_a = 0.90\ (CBD\ location)$$

The remainder of the adjustment factors vary by lane group. The parking and bus blockage will be 1.000 for the left-only and through-only lane groups and will be calculated for the right-only lane group as follows:

$$f_p = \frac{N - 0.10 - \left(\frac{18N_m}{3600}\right)}{N} = \frac{1 - 0.10 - \left(\frac{18*15}{3600}\right)}{1} = 0.993$$
$$f_{bb} = \frac{N - \left(\frac{14.4N_B}{3600}\right)}{N} = \frac{1 - \left(\frac{14.4*20}{3600}\right)}{1} = 0.920$$

The lane utilization factor will be 1.00 for the single left-only and right-only lane groups.
From Table 22-7, the through lane group lane utilization factor is:

f_{LU} = 0.952 (default value, Table 22-7)

The adjustment factor for right turns will be applied only the exclusive right-turn lane group, as follows:

$$f_{RT} = 0.85$$

The adjustment for left turns will only be applied to the exclusive left-turn lane:

f_{LT} = 0.95

The estimation of f_{Rpb} and f_{Lpb} involves several steps as follows. The effective flow rate for pedestrians in both crosswalks (inferfering with right and left turns respectively) is given by:

$$v_{pedg} = v_{ped}\left(C/g_p \right) = 100 * \left(100/60\right) = 166.7 \ \ peds/hg$$

The occupancy of both crosswalks may then be determined as:

$$OCC_{pedg} = \frac{v_{pedg}}{2000} = \frac{166.7}{2000} = 0.0834$$

The conflict zone occupancy rate for both movements is:

$$OCC_r = OCC_{pedg} = 0.0834$$

Assuming that more than one lane is available for left- and right-turning vehicles to turn into, then the adjustment factor during the permitted portion of the phase is given by:

$$A_{pbT} = 1 - 0.6OCC_r = 1 - \left(0.6 * 0.0834\right) = 0.950$$

This value holds for both left- and right-turns. Then:

$$f_{Rpb} = A_{pbt} = 0.950$$
$$f_{Lpb} = A_{pbt} = 0.950$$

The saturation flow rate may now be estimated using Eqn 22-11:

	s_o pc/hg/ln	N	f_w	f_{HVg}	f_p	f_{bb}	f_a	f_{LU}	f_{RT}	f_{LT}	f_{Rpb}	f_{Lpb}	f_{wz}		s vph
L	1900	1	1.00	0.894	1.00	1.00	0.90	1.00	1.00	0.95	1.00	0.95	1.00		1380
T	1900	2	1.00	0.894	1.00	1.00	0.90	0.952	1.00	1.00	1.00	1.00	1.00		2911
R	1900	1	1.00	0.894	0.825	0.920	0.90	1.00	0.85	1.00	0.95	1.00	1.00		1127

The capacity of each lane group is found using Equation 22-2:

$$c = s\left(\frac{g}{C}\right)$$

Lane Group	*s* (vph)	*g* (s)	*C* (s)	*c* (vph)
L	1380	60	100	828
T	2911	60	100	1747
R	1127	60	100	676

Problem 22-2
All relevant parameters for the Inputs are specified in the problem statement.

Convert Demand Volumes to Demand Flow Rates

To adjust the volume, hourly demand volumes are adjusted to reflect peak flow rates using the Peak Hour Factor. Lane groups for analysis are established.

In this case, there will be six lane groups for analysis. EB traffic is in two lane groups. There is a EB right-turn lane, which must be treated as a separate lane group and two through lanes, which will be analyzed as one lane group. There is a WB left-turn lane, and a through lane, each of which must be treated as a separate lane group. On the NB stem of the T-intersection, there are exclusive LT and RT lanes that must also be treated separately. Hourly volumes are adjusted to peak flow rates as follows:

$$v = \frac{V}{PHF}$$

The computations for peak flow rate are shown in Table A.

Table A: Computations for the Volume Adjustment Module

Approach	Mvt	Vol (veh/h)	PHF	Lane Group Flow Rate (veh/h)
EB	T	525	0.92	571
	R	40	0.92	43
WB	L	200	0.92	217
	T	600	0.92	652
NB	L	400	0.92	435
	R	330	0.92	359

Saturation Flow Analysis

Saturation flow rates are given in the problem statement.

Capacity Analysis

The results of volume and saturation flow rate computations are combined to determine v/s ratios, critical phases and the sum of critical lane v/s ratios, v/c ratios, and the critical v/c ratio. Capacities are computed as:

$$c = s\left(\frac{g}{C}\right)$$

where effective green times are equal to actual green times using the default values for ℓ_1 and *e* (2.0 s each).

The g/C ratios may be computed from the phase timings given in the problem statement as follows:

$$(g/C)_{EB/WB} = \frac{39}{70} = 0.557$$

$$(g/C)_{NB} = \frac{22.5}{70} = 0.321$$

Saturation flow rates are provided in the problem. Capacity analysis computations are carried out in the table that follows.

Table B: Capacity Analysis Computations

Lane Group	Vol (veh/h)	Sat Flow Rate (veh/hg/ln)	v/s Ratio	g/C Ratio	Capacity c (veh/h)	v/c Ratio (X)
EB TH	571	1680	0.170	0.557	1872	0.305
EB RT	43	1500	0.029	0.557	836	0.052
WB LT	217	774	0.269	0.557	431	0.483
WB TH	652	1697	0.384	0.557	945	0.690
NB LT	435	1676	0.259	0.321	539	0.807
NB RT	359	1500	0.239	0.321	482	0.744

As there are only two phases, the critical movement in Phase A is the WB through, with a v/s ratio of 0.384. In Phase B, the critical movement is the NB left, with v/s ratio of 0.259 The sum of the critical v/s ratios is 0.384 + 0.259 = 0.643

The critical v/c ratio, X_c, is computed as:

$$X_c = \sum (v/s)_{ci} * \left(\frac{C}{C-L} \right)$$

Lost times are computed as:

$$t_{Li} = \ell_1 + (Y - e)$$

$$t_{LA} = 2.0 + 4.0 - 2.0 = 4.0\ s$$
$$t_{LB} = 2.0 + 4.5 - 2.0 = 4.5\ s$$

L = 4.0+4.5 = 8.5 s.

$$X_c = 0.643 * \left(\frac{70}{70 - 8.5} \right) = 0.733$$

Problem 22-3
The WB approach has two lane groups: a through-only lane group and an exclusive left-turn lane group.

1. The delay for the through lane may be found using Webster's equation.

   From Problem 22-2, *g/C* = 0.557; *v/s* = 0.384; *X* = 0.69
   For arrival type 5, *P* = (5/3)*(0.557) = 0.93
   Find progression factor, *PF*

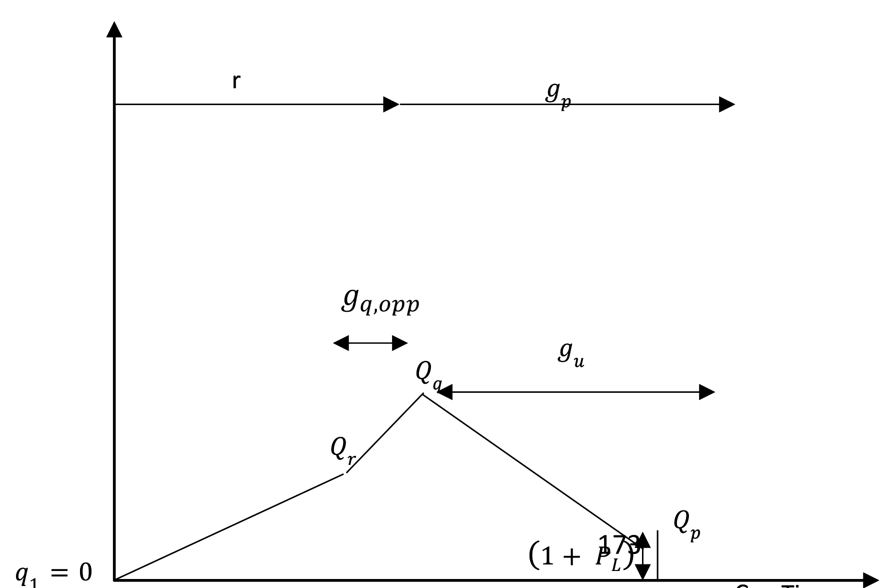
$$PF = \frac{1-P}{1-g/C} * \frac{1-y}{1-\min(1,X)P} * \left[1 + y\frac{1-PC/g}{1-g/C}\right]$$
$$PF = \frac{1-0.93}{1-0.557} * \frac{1-0.384}{1-0.69*0.93} * \left[1 + 0.384\frac{1-0.93*70/39}{1-39/70}\right] = 0.11$$

$$d_1 = \frac{0.5C[1-g/C]^2}{1-[\min(1,X)]*g/C} * PF = \frac{0.5*70(1-0.557)^2}{1-0.69*0.557} * 0.11 = 1.27\ s/veh$$
$$d_2 = 900T\left[(X-1) + \sqrt{(X-1)^2 + \left(\frac{8kIX}{cT}\right)}\right] = 4.12\ \text{sec}/veh$$
$$d = d_1 + d_2 = 1.27 + 4.12 = 5.39\ s/veh$$

   LOS = A for WB through lane

2. The delay for the left-turn lane group moves in permitted mode and must be found using the IQA method.

   The QAP for the westbound left-turn lane group is shown below. Notice that because it is an exclusive left-turn lane, there is no g_f period; the first vehicle is always a left-turning vehicle, so $g_f = 0$



The proportion of vehicles that arrive on green with an arrival type of 5 is found using Equation 22-8:

$$P = (5/3)*(39/70) = 0.928$$

The arrival rate on red and green, respectively, are found:

$$v_r = \frac{(1-P)VC}{r} = \frac{(1-0.928)217*70}{31} = 35veh/h$$
$$v_g = \frac{PVC}{g} = \frac{0.928*217*70}{39} = 362veh/h$$

1. Starting at the beginning of effective red, set the initial queue to zero, q_1 = 0.
2. Find the queue at the end of effective red:
3.

$$q_2 = q_1 + \left(\frac{v_r - s}{3600}\right)*r$$
$$Q_r = 0 + \frac{35-0}{3600}*31 = 0.30\ vehs$$

Note that you would expect a very small queue at the end of red, given that your arrival type is excellent (AT=5) and most vehicles are arriving on green.

4. Incremental delay during effective red is

$$d_r = r*\left(\frac{q_1+q_2}{2}\right)$$
$$d_r = 31*\frac{0+0.30}{2} = 4.68$$

5. Calculate the time until the opposing queue clears, $g_{q,opp}$

P (for arrival type 2) = (2/3) * 0.56 = 0.37

$$v_{r,opp} = \frac{(1-P)V_{opp}C}{r} = \frac{(1-0.37)285*70}{31} = 405veh/h$$
$$v_{g,opp} = \frac{PVC}{g} = \frac{0.37*285*70}{39} = 190veh/h$$

$$g_{q,opp} = \frac{v_{r,opp}*r}{s_o - v_{g,opp}} = \frac{405*31}{1680-190} = 8.43s$$

6. Calculate subject queue (WB left-lane queue) at the end of $g_{q,opp}$

$$Q_{gq,opp} = Q_r + \left(\frac{v_g - s}{3600}\right) = 0.30 + \left(\frac{362 - 0}{3600}\right) = 2.1\ vehs$$

7. Find delay during $g_{q,opp}$

$$d_{gq,opp} = g_{gq,opp} * \left(\frac{Q_r + Q_{gq,opp}}{2}\right) = 8.43 * \left(\frac{0.30 + 2.1}{2}\right) = 10.2\ veh - s$$

8. Find Unsaturated green time, g_u

$$g_u = 39 - 8.43 = 30.57\ s$$

9. Find the left-turn thru-car equivalency, E_{L1}

$$E_{L1} = \frac{s_o}{s_p} = \frac{1697}{774} = 2.19$$

10. Saturation Flow rate during g_u

$$s_{gu} = s_{TH} * \frac{1}{1 + (E_{L1} - 1)} = 1697 * \frac{1}{2.19} = 774$$

11. Calculate queue at end of g_u

$$Q_{gu} = Q_{gq,opp} + \left(\frac{v_g - s_{gu}}{3600}\right) * g_u = 2.1 + \frac{362 - 774}{3600} * 30.57 = \text{-}1.38$$

12. Since the number is negative, we find the time during the green phase when the queue clears, Δt_2 :

$$\Delta_{t2} = \frac{3600 * Q_{gq,opp}}{v_g - s} = \frac{3600 * 2.1}{774 - 362} = 18.5\ s$$

13. Find delay during g_u

$$d_{gu} = \Delta_{t2} * \frac{Q_{gq,opp} + 0}{2} = 19.5 veh - s$$

14. Find number of vehicles arriving on green

$$n_a = \frac{v_g}{3600} * g = \frac{362}{3600} * 39 = 3.93\ vehs$$

15. Find uniform delay, d_1

$$d_1 = \frac{d_r + d_{gq,opp} + d_{gu}}{Q_r + n_a} = \frac{4.68 + 10.2 + 19.5}{0.30 + 3.93} = 8.14\ sec/veh$$

16. Find Incremental delay, d_2

$$d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\left(\frac{8kIX}{cT}\right)}\right] = 2.4 \sec/veh$$

The value of T was set at 0.25 h (15 m), the standard analysis period. The value of “k” for pretimed signals is 0.50, and the value of “I” for isolated signal analysis is always 1.0. Values for X and c are found from Problem 22-2.

$$d = d_1 + d_2 = 8.14 + 2.4 = 10.54\ s/veh$$

Since the cutoff for *LOS* A = 10 s; this lane group *LOS* = B

Problem 22-4

Lane Group Delay Computations

Item	Lane Group					
	EB T	**EB R**	**WB LT**	**WB TH**	**NB LT**	**NB RT**
C	80	80	80	80	80	80
X	0.941	0.159	0.359	0.627	0.960	0.886
d_1	**17.3**	**13.4**	**0.6**	**0**	**4.35**	**4.21**
T	0.25	0.25	0.25	0.25	0.25	0.25
c	607	271	605	1040	453	405
d_2	**24.5**	**1.2**	**1.7**	**2.9**	**33.5**	**23.7**
d_3	**0**	**0**	**0**	**0**	**0**	**0**
d	**41.8**	**14.6**	**2.2**	**2.9**	**37.8**	**27.9**

Volumes per lane group may be calculated using $X = v/c$ and solving for v

Item	Lane Group					
	EB T	**EB R**	**WB LT**	**WB TH**	**NB LT**	**NB RT**
c	607	271	605	1040	453	405
X	0.941	0.159	0.359	0.627	0.960	0.886
v	**571**	**43**	**217**	**652**	**435**	**359**

The approach weighted average delays are:

$$d_{EB} = \frac{(571*41.8)+(43*14.6)}{(571+43)} = 39.9\ s/veh$$

$$d_{WB} = \frac{(217*2.2)+(652*2.9)}{(217+652)} = 2.7\ s/veh$$

$$d_{NB} = \frac{(435*37.8)+(359*27.9)}{(435+359)} = 33.3\ s/veh$$

The overall intersection delay is:

$$d_I = \frac{(614*39.9)+(869*2.7)+(854*33.3)}{(614+869+854)} = 23.4\ s/veh$$

From the LOS Table 22-1, the following levels of service apply:

- EB T — LOS D
- EB R — LOS B
- WB LT — LOS A
- WB TH — LOS A
- NB LT — LOS D
- NB RT — LOS C
- EB Approach — LOS D
- WB Approach — LOS A
- NB Approach — LOS C
- Intersection — LOS C

While the intersection delay of 23.4 s/veh is acceptable, there is considerable variation in the lane group v/c and delay values which might be improved by retiming the signal.

Problem 22-5

Vehicle in queue	Cycle 1	Cycle 2	Cycle 3	Cycle 4	Cycle 5	Sum of Sat Hdwys	No. of Sat hdwys
1	2.8	2.9	3.0	3.1	2.7	**0**	
2	2.6	2.6	2.5	3.5H	2.6	**0**	
3	3.9L	2.3	2.2	2.9	2.5	**0**	
4	10.2H	2.1	2.0	2.5	2.0	**0**	
5	8.7	4.0L	1.9	2.2	1.9	**18.7**	**5**
6	3.0	9.9L	2.2	2.0	1.9	**19**	**5**
7	<u>2.9</u>	9.8	2.9H	1.9	3.6H	**21.1**	**5**
8	5.0	3.3	2.6	<u>1.8</u>	9.0	**16.7**	**4**
9	7.1	2.8	<u>2.1</u>	7.0	<u>4.0</u>	**8.9**	**3**
10	9.0	2.2	4.0	8.0	4.9	**2.2**	**1**
11		<u>1.9</u>	5.0		9.0	**1.9**	**1**
12		5.5				**0**	**0**
13		4.0				**0**	**0**
SUM						**88.5**	**24**

Prevailing saturation headway:

$$h = \frac{88.5}{24} = 3.7\frac{s}{veh}$$

$$s = \frac{3600}{3.7} = 973\ vphgpl$$

Base saturation headway:

$$h = \frac{7.9}{4} = 2.0\frac{s}{veh}$$

$$s = \frac{3600}{2} = 1800\ vphgpl$$

Solutions to Problems in Chapter 23

Planning Level Analysis of Signalized Intersections

Problem 23-1

The left-turn check is only needed for the westbound lefts. There are no EB left turns. The NB lefts are not opposed, and thus behave as protected lefts. The check for the WB lefts results in the WB left turns needing a protected phase.

Left-turn Check for WB Lefts

Left-Turn Check Number	Westbound
Check #1	No
Check #2	No
Cross-Product	210*510=107,100
Threshold	> 90,000
Check #3	Yes
Final Decision	Protected

Adjustment of Demand Volumes

	Eastbound			Westbound			Northbound		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
Demand Volume,vph	0	510	45	210	500	0	400		380
E_{HV}		1.05	1.05	1.05	1.05		1.05		1.05
E_{PHF}		1.14	1.14	1.14	1.14		1.14		1.14
E_{RT}		1	1.3	1	1		1		1.5
E_{LT}		1	1	1.05	1		1.05		1
E_P		1	1	1	1		1		1
E_{LU}		1.1	1.1	1.1	1.1		1		1.05
E_{Other}		1	1	1	1		1		1
Adjusted Flow Rate, tcu/hr		669	77	263	626		501		680
Lane Group Flow Rate, tcu/hr		746		263	626		501		680
Lane Group Flow Rate per Lane, tcu/hr/ln		373		263	626		501		680
Critical Lanes				X	X				X

Number of Critical Phases	3
Sum of Critical Lane Flow Rates	1570
Cycle Length	80
Intersection Capacity	1615
X_c	0.97
Intersection Sufficiency Status	NEAR

	EB	WB		NB	
Lane Group	TR	L	TR	L	R
Lane Group Flow Rate per Lane, tpc/hr/ln	373	263	626	501	680
Critical Lanes		X	X		X
g/C	0.25	0.31	0.25	0.29	0.29
Lane Group Capacity	950	593	475	546	546
v/c ratio	0.79	0.44	1.32	0.92	1.25
Uniform Delay, d_1	28.0	21.9	30.0	27.6	28.5
Progression Factor	0.7	1.25	1.25	1.0	1.0
Incremental Delay, d_2	6.5	2.4	157.7	22.7	125.0
Control Delay, s/veh	26.1	29.8	195.2	50.3	153.5
Lane Group LOS	C	C	F	D	F
Approach Delay	26.1	146.3		109.7	
Approach LOS	C	F		F	

Intersection Delay	91.5		
Intersection LOS	F		
Intersection Capacity, tcu/hr/ln	1615	Intersection v/c	0.97

Obviously this intersection is not performing well with the given splits of green times. The intersection v/c is less than one, so with retiming of the phases, improvement should be seen. If the green times are divided in proportion to the critical phases, the following results.

	EB	WB		NB	
Lane Group	TR	L	TR	L	R
Lane Group Flow Rate per Lane, tcu/hr/ln	373	263	626	501	680
Critical Lanes		X	X		X
g/C	0.34	0.14	0.34	0.72	0.97
Lane Group Capacity	1289	270	645	700	700
v/c ratio	0.58	0.97	0.97	0.92	1.25
Uniform Delay, d_1	21.7	34.1	26.1	21.7	24.9
Progression Factor	0.7	1.25	1.25	1.0	1.0
Incremental Delay, d_2	1.9	47.8	29.2	6.2	27.8
Control Delay, s/veh	17.1	90.7	61.8	27.9	52.7
Lane Group LOS	B	F	E	C	D
Approach Delay	17.1	70.3		42.2	
Approach LOS	B	E		D	

Intersection Delay	46.0		
Intersection LOS	D		
Intersection Capacity, tpc/hr/ln	1615	Intersection v/c	0.97

Problem 23-2

Left-turn Check Results

Left-Turn Check Number	Eastbound	Westbound	Northbound	Southbound
Check #1	No	No	No	No
Check #2	No	No	No	Yes
Cross-Product	45*500=22,500	40*450=18,000	125*700=87,500	250*900=225,000
Threshold	> 90,000	> 90,000	> 110,000	> 110,000
Check #3	No	No	No	Yes
Final Decision	Permitted	Permitted	Protected	Protected

Adjustment of Demand Volumes

	Eastbound			**Westbound**			**Northbound**			**Southbound**		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
Demand Volume,vph	99	1850	50	40	1100	120	125	950	95	200	1350	100
E_{HV}	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03
E_{PHF}	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
E_{RT}	1	1	1.2	1	1	1.2	1	1	1.2	1	1	1.2
E_{LT}	5	1	1	5	1	1	1.05	1	1	1.05	1	1
E_{P}	1	1	1	1	1	1	1	1	1	1	1	1
E_{LU}	1.1	1.1	1.1	1.1	1.1	1.1	1	1.1	1.1	1	1.1	1.1
E_{Other}	1	1	1	1	1	1	1	1	1	1	1	1
Adjusted Flow Rate, tpc/hr	610	2278	74	246	1355	177	147	1170	140	235	1663	148
Lane Group Flow Rate, tpc/hr		2962			1778		147	1310		235	1810	
Lane Group Flow Rate per Lane, tpc/hr/ln		987			593		147	437		235	603	
Critical Lanes		X								X	X	

Number of Critical Phases	3
Sum of Critical Lane Flow Rates	1826
Cycle Length	90
Intersection Capacity	1647
X_c	1.109
Intersection Sufficiency Status	OVER

Solutions to Problems in Chapter 24

Urban Streets and Arterials – Complete Streets Approaches

Problem 24-1

Urban street design has changed from almost exclusively designing for automobiles to move efficiently through the facility to designing for all modes to have equal importance in the planning and design of the facility. With this type of design comes many benefits. The streets are safer because vehicles are moving slower, pedestrians and bicycles are given defined areas on the street, often separated from the vehicles. Because of the separation and increased feeling of safety, more pedestrians and bicycles are encouraged, which leads to better health and improved air quality. The savings to individuals by switching to transit, bicycle riding, and walking gives them more money to spend in other ways, which can then be used in the local economy. Businesses get the benefit of better access for bicycles and pedestrians. Adding improved access to communities leads to more building and private investment in that area, which in turn leads to more jobs and increased property values.

Problem 24-2

The factors that affect level of service for pedestrians and why each is important include:

a. the width of the sidewalk for more comfort and increased walking speed,
b. the existence of a bicycle lane and the width of the shoulder and any buffers, which by adding more separation from the moving vehicles provide more safety and thus comfort for the pedestrian
c. Average vehicle speed because the faster the vehicles move, the less safe pedestrians feel
d. At the intersection, the delay experienced by the pedestrian affects their quality of service, as well as the vehicle demand and speed, which affect their crossing comfort

Problem 24-3

The factors that affect level of service for bicycles and why each is important include:

a. Is there a bike lane, which gives dedicated space to bicycles
b. Vehicle demand and speed, which affects the comfort of bicycles sharing the same street
c. Percent heavy vehicles because heavy vehicles are wider
d. Shoulder width, outside lane width and other buffer widths that separate the bicycles from the moving vehicles providing more safety for the bicyclists

e. Unsignalized intersections and access points, which cause more conflicts for the bicycles

Problem 24-4

The factors that affect level of service for public transit and why each is important are:

a. The speed of the transit vehicles
b. Delay at the intersection
c. Perceived travel time
d. Perceived waiting time at the transit stop

The first two factors (speed and delay) measure the time that the trip takes. The perceived travel time represents how the passenger experiences this travel time, that includes how many times the vehicle must stop and for how long each stop is. The perceived waiting time takes into account how long they must wait for the vehicle, but also how comfortable is that wait: are there benches and is there shelter.

Problem 24-5

Level of service for automobiles is defined by the average travel speed on the facility. The level of service thresholds are different for different types of arterials, such as major versus minor arterials. The type of arterial is characterized by its base free flow speed.

Drivers however do not perceive their quality of service by average travel speed. For instance, on a facility with a based free flow speed of 40 mph, level of service A is defined as speeds greater than 32 mph, LOS B is greater than 27 mph, LOS C is greater than 20 mph. These differences in speed are not as important to drivers as the number of stops they experience or having a separate left-turn bay, for example. Thus a separate LOS score is defined for automobile quality of service.

Solutions to Problems in Chapter 25

Unsignalized Intersections and Roundabouts

Problem 25-1

The intersection under study is a T-intersection with an exclusive LT lane on the major street. These two factors will simplify the analysis, as several movements at a typical four-leg TWSC intersection do not exist. The computational steps outlined in the text will be followed.

Step 1 – Express Demand as Flow Rate During a Peak 15-Minute Analysis Period

This step is not necessary, as all of the demands are stated as peak 15-minute flow rates. Note that all computations are carried out in *vehicles per hour*. There are no conversions to passenger car equivalents.

Step 2 – Determine the Conflicting Flow Rates for Each Movement

There are only six vehicular movements at the T-intersection. Using the movement numbering scheme of Figure 25-1, they are Mvts 2, 3, 4, 5, 7, and 9. Pedestrian flows 13, 14, and 15 also exist.

Major street through flows (2, 5) do not face any conflicting movements, nor does the major street right turn (3). Using the equations given in Table 25-2, the following conflicting flow rates are computed for the three conflicted movements (4,7,9). Note that Mvt 7 is a one-stage movements, and the total conflicting flow rate is the sum of the conflicting flow rates for Stages I and II.

$$v_{c4} = v_2 + v_3 + v_{15} = 250 + 25 + 30 = 305 \ \ conflicts/h$$
$$v_{c9} = v_2 + 0.5v_3 + v_{14} + v_{15} = 250 + (0.5*25) + 40 + 30 = 332.5 \Rightarrow 333 \ conflicts/h$$
$$v_{c7I} = 2v_1 + v_2 + 0.5v_3 + v_{15} = (2*0) + 250 + (0.5*25) + 30 = 292.5 \Rightarrow 293 \ conflict/h$$
$$v_{c7II} = 2v_4 + v_5 + 0.5v_6 + 0.5v_{12} + 0.5v_{11} + v_{13} = (2*20) + 200 + (0.5*0) + (0.5*0) + (0.5*0) + 45 = 285 \ conflicts/h$$
$$v_{c7} = v_{c7I} + v_{c7II} = 293 + 285 = 578 \ conflicts/h$$

Step 3 – Determine the Critical Gaps (Headways) and Follow-Up Times

Critical gaps (headways) are computed using Equation 25-2:

$$t_{ci} = t_{cbase} + f_{cHV}P_{HV} + f_{cG}G - f_{3LT}$$

Note that there are 5% trucks in all movements, and that the NB grade is +2%. The WB grade may be assumed to be level. Base critical gaps are selected from Table 25-1 for a 4-lane major street. Adjustment factors are taken from Table 25-2. Note also that P_{HV} is expressed as a decimal (0.05), while G is expressed as a percent.

$$t_{c4} = 4.1 + (2.0*0.05) + (0*0) - 0 = 4.2\ s$$
$$t_{c7} = 7.5 + (2.0*0.05) + (0.2*2) - 0.7 = 7.3\ s$$
$$t_{c9} = 6.9 + (2.0*0.05) + (0.1*0) - 0 = 7.0\ s$$

Follow-up times are computed using Equation 25-3:

$$t_{fi} = t_{fbase} + f_{fHV} P_{HV}$$

Base follow-up times are taken from Table 25-1, and adjustments are taken from Table 25-2.

$$t_{f4} = 2.2 + (1.0*0.05) = 2.25\ s$$
$$t_{f7} = 3.5 + (1.0*0.05) = 3.55\ s$$
$$t_{f9} = 3.3 + (1.0*0.05) = 3.35\ s$$

Step 4 – Compute Potential Capacities

Potential capacities are computed using Equation 25-4.

$$c_{pi} = v_{ci}\left[\frac{e^{-v_{ci} t_{ci}/3600}}{1-e^{-v_{ci} t_{fi}/3600}}\right]$$
$$c_{p4} = 305\left[\frac{e^{-305*4.2/3600}}{1-e^{-305*2.25/3600}}\right] = 1{,}231\ veh/h$$
$$c_{p7} = 578\left[\frac{e^{-578*7.3/3600}}{1-e^{-578*3.55/3600}}\right] = 412\ veh/h$$
$$c_{p9} = 333\left[\frac{e^{-333*7.0/3600}}{1-e^{-333*3.35/3600}}\right] = 654\ veh/h$$

Step 5 – Determine Movement Capacities

Movement capacities involve determining the impeding effects of other vehicular and pedestrian flows that may consume some of the available gaps for the subject movement. These impedances are summarized in Table 25-4:

- Movement 4 is impeded by pedestrian movement 15.
- Movement 9 is impeded by pedestrian movements 14 and 15.
- Movement 7 is impeded by vehicular movements 1, 4, 11, and 12, and by pedestrian movements 13 and 15. Because this is a T-intersection, vehicular movements 1, 11, and 12 do not exist, so only vehicular movement 4 impedes movement 7.

Because of the sequence of priorities, movement capacities are determined in the following sequence: 4, 9, 7.

Movement Capacity of Movement 4

Pedestrian movement 15 is the only one that impedes vehicular movement 4. The impedance factor for pedestrian movement 15 is given by Equation 25-6:

$$P_j = 1 - \frac{v_j \left({w}/{S_p} \right)}{3600}$$

$$P_{15} = 1 - \frac{30 \left({10}/{3.5} \right)}{3600} = 0.976$$

Then, the movement capacity of Movement 4 is:

$$c_{m4} = c_{p4} P_{15} = 1231 * 0.976 = 1{,}201 \; veh/h$$

Movement Capacity of Movement 9

Movement 9 is impeded by pedestrian movements 14 and 15. The impedance factor for pedestrian movement 15 has already been computed (0.976). The impedance factor for pedestrian movement 14 is found using Equation 25-6:

$$P_{14} = 1 - \frac{40 \left({10}/{3.5} \right)}{3600} = 0.968$$

The movement capacity for Movement 9 is:

$$c_{m9} = c_{p9} * P_{14} * P_{15} = 654 * 0.968 * 0.976 = 618 \; veh/h$$

Movement Capacity of Movement 7

Movement 7 is impeded by vehicular movement 4 and pedestrian movements 13 and 15. P_{15} has already been computed (0.976).

The impedance factor for vehicular movement 4 is given by Equation 25-5:

$$P_y = 1 - \frac{v_y}{c_{my}} = 1 - \frac{20}{1201} = 0.983$$

The impedance factor for pedestrian movement 13 is given by Equation 25-6:

$$P_{13} = 1 - \frac{45 \left({10}/{3.5} \right)}{3600} = 0.964$$

Then, the movement capacity for Movement 7 is:

$$c_{m7} = c_{p7} P_4 P_{13} P_{15} = 412 * 0.983 * 0.964 * 0.976 = 381\ veh/h$$

Step 6 – Determine Shared-Lane Capacities

Movements 7 and 9 share a single lane on the STOP-controlled approach. The shared-lane capacity for this lane is found using Equation 25-16:

$$c_{SHx} = \frac{\sum v_i}{\sum \left(v_i / c_{mi} \right)} = \frac{100 + 125}{\left(100/381\right) + \left(125/619\right)} = \frac{225}{0.262 + 0.202} = 485\ veh/h$$

Step 7 – Estimate Delays and Determine LOS

Delays must be computed for Movement 4, which has an exclusive lane, and for the shared lane serving Movements 7 and 9. The delay is computed using Equation 25-17:

$$d_x = \left(\frac{3600}{c_{mx}}\right) + 900T \left[\left(\frac{v_x}{c_{mx}} - 1\right) + \sqrt{\left(\frac{v_x}{c_{mx}} - 1\right)^2 + \frac{\left(\frac{3600}{c_{mx}}\right)\left(\frac{v_x}{c_{mx}}\right)}{450T}} \right] + 5$$

$$d_4 = \left(\frac{3600}{1201}\right) + 900 * 0.25 \left[\left(\frac{20}{1201} - 1\right) + \sqrt{\left(\frac{20}{1201} - 1\right)^2 + \frac{\left(\frac{3600}{1201}\right)\left(\frac{20}{1201}\right)}{450 * 0.25}} \right] + 5 = 8.1\ s/veh$$

$$d_{SH7,9} = \left(\frac{3600}{485}\right) + 900 * 0.25 \left[\left(\frac{225}{485} - 1\right) + \sqrt{\left(\frac{225}{485} - 1\right)^2 + \frac{\left(\frac{3600}{485}\right)\left(\frac{225}{485}\right)}{450 * 0.25}} \right] + 5 = 18.7\ s/veh$$

Levels of service are defined in Table 25-7. Movement 4 (major street LT) operates at LOS A, while the shared lane for Movements 7 and 9 operates at LOS C. Both are these are acceptable for this type of control. While an average delay for the intersection as a whole could be computed, it would have little meaning, given that many vehicles experience no delay. The LOS for the shared lane, which is the one controlled by a STOP sign, is the most important indicator of how well the situation is working.

<u>Problem 25-2</u>

This is a very simple situation, which it must be to allow a hand-based (aided with spreadsheets) solution.

There are two single-lane STOP-controlled approaches: EB and NB. Each will be considered as a “subject” approach. In each case, there are only two possible scenarios for vehicles on the other approach: either a vehicle is present, or no vehicle is present. These scenarios are detailed in the table that follows:

Scenarios for Problem 25-2

Subject Approach	Scenario	Vehicles in Conflicting Lane	Geometry Group (Table 25-9)	DOC
EB	1	0	1	1
	2	1	1	3
NB	1	0	1	1
	2	1	1	3

Note that Degree of Conflict (DOC) 1 exists when the only vehicle present is on the subject approach lane. DOC 3 exists when there is a vehicle on the subject approach lane and *one* conflicting approach lane.

Step 1 – Convert Volumes to Flow Rates
This step is not needed, as demand are already expressed as flow rates.

Step 2 - Determine the Intersection Geometry Group
Table 25-9 indicates that this intersection may be classified in Geometry Group 1.

Step 3 - Determine Saturation Headways for Each Scenario
For the two subject approaches, there are a total of 4 scenarios, two for each subject approach. Saturation headways for each scenario are estimated using Equation 25-19 and 25-20:

$$h_{si} = h_{basei} + h_{adj}$$

$$h_{adj} = h_{LT} P_{LT} + h_{RT} P_{LT} + h_{HV} P_{LT}$$

Base saturation headways (h_{basei}) are found in Table 25-10. Adjustment factors (h_j) are taken from Table 25-11. Values for P_{LT}, P_{RT}, and P_{HV} are given in the problem statement, but must be expressed as a decimal for use.

From Table 25-10, base saturation headways for Scenario 1 (for both approaches) is 3.9 s/veh (DOC 1, Group 1). For both Scenarios 2, the value is 5.8 s/veh (DOC 3, Group 1, 1 vehicle present on conflicting approach). From Table 25-11, h_{LT} = 0.2, h_{RT} = -0.6, and h_{HV} = 1.7. From the problem statement, P_{LT} = 0.00 (NB) and 0.05 (EB), P_{RT} = 0.10 (NB) and 0.00 (EB). The proportion of heavy vehicles, P_{HV} = 0.05 (NB) and 0.08 (EB). Then:

$$h_{adjNB} = (0.2*0.00) - (0.6*0.10) + (1.7*0.05) = 0.025$$
$$h_{adjEB} = (0.2*0.05) - (0.6*0.00) + (1.7*0.08) = 0.146$$
$$h_{sNB1} = 3.9 + 0.025 = 3.925\ s/veh$$
$$h_{sNB2} = 5.8 + 0.025 = 5.825\ s/veh$$
$$h_{sEB1} = 3.9 + 0.146 = 4.046\ s/veh$$
$$h_{sEB2} = 5.8 + 0.146 = 5.946\ s/veh$$

Step 4: Determine the Departure Headway for Each Approach
There are two scenarios for each subject approach (NB, EB). The departure headways depend upon the degree of saturation (*X*) for the conflicting approach in each case. The process is iterative, but starts with an assumption that all values of h_d are 3.2 s/veh. Then, using Equation 25-22:

$$X_j = \frac{v_j h_{dj}}{3600}$$
$$X_{NB} = \frac{325*3.2}{3600} = 0.289$$
$$X_{EB} = \frac{300*3.2}{3600} = 0.267$$

These values set the probabilities that the conflicting lane is empty or occupied. For the NB approach, it 0.267 probable that the lane is occupied, and (1-0.267) = 0.733 probable that it is empty. For the EB approach, it is 0.289 probable that the lane is occupied, and (1-0.289) = 0.711 probable that it is empty. The “empty” states exist for both Scenarios 1, while the “occupied” states exist for both Scenarios 2. Since there is only one conflicting lane to consider in each case, there are no multiple probabilities to multiply. Thus, the probabilities that each scenario exists are as follows:

$$P_{NB1} = 0.733$$
$$P_{NB2} = 0.267$$
$$P_{WB1} = 0.711$$
$$P_{WB2} = 0.289$$

Note that the EB approach is the “conflicting approach” for the NB subject approach, and vice-versa.

Equations 25-25 must be used to adjust these computations. The probability that DOC 1 exists is the probability of each Scenario 1 – the conflicting lane is empty. The probability that DOC 3 exists is the probability of each Scenario 2 – one conflicting lane is occupied. The probability of all other DOC’s (2, 4, and 5) is 0.0, as none of these can occur.

The adjustment to initial scenario probabilities is estimated using Equations 25-25:

$$AdjP_{DOC1} = 0.01[P_{DOC2} + 2P_{DOC3} + 3P_{DOC5}]/1$$
$$AdjP_{DOC1-NB} = 0.01[0.0 + (2*0.267) + (3*0)]/1 = 0.0053\, s/veh$$
$$AdjP_{DOC1-EB} = 0.01[0.0 + (2*0.289) + (3*0)]/1 = 0.0058\ s/veh$$
$$AdjP_{DOC3} = 0.01[P_{DOC4} + 2P_{DOC5} - 3P_{DOC3}]/6$$
$$AdjP_{DOC3-NB} = 0.01[0.0 + (2*0.0) - (3*0.267)]/6 = -0.0013\ s/veh$$
$$AdjP_{DOC3-WB} = 0.01[0.0 + (2*0.0) - (3*0.289)]/6 = -0.0014 s/veh$$

Adjustments for DOC 1 are applied to Scenarios 1, while adjustments for DOC 3 are applied to Scenarios 2. Then:

$$P_i^{'} = P_i + AdjP_i$$
$$P_{NB1}^{'} = 0.733 + 0.0053 = 0.7383$$
$$P_{NB2}^{'} = 0.267 - 0.0013 = 0.2657$$
$$P_{EB1}^{'} = 0.711 + 0.0058 = 0.7168$$
$$P_{EB2}^{'} = 0.289 - 0.0014 = 0.2876$$

Departure headways may now be computed using Equation 25-21:

$$h_d = \sum_i P_i^{'} h_{si}$$
$$h_{NB} = (0.7383*3.925) + (0.2657*5.825) = 2.898 + 1.548 = 4.446 \Rightarrow 4.4\ s/veh$$
$$h_{EB} = (0.7168*4.046) + (0.2876*5.946) = 2.900 + 1.710 = 4.61 \Rightarrow 4.6\, s/veh$$

This, however, is not the final result. The computed values (4.4 and 4.6 s/veh) are quite different from the initial assumed value of h_d (3.2 s/veh). The result must now be iterated until the assumed and computed values agree to within ±0.1. Each successive iteration begins with the results from the previous iteration. For the NB and EB subject approaches, the results of these iterations (each of which follows the same steps as the initial computation) are shown in table that follows.

As can be seen, in each case, three iterations closes the equations, and the departure headways are 4.7 s/veh for the NB approach and 4.1 s/veh for the EB approach.

Iterated Solutions for Departure Headway for the Sample AWSC Intersection

Northbound Solution

Iteration No:	1	2	3
Demand Flow, NB	325	325	325
Demand Flow, EB	300	300	300
Initial h_d	**3.2**	**4.4**	**4.8**
h_{base} *(Scenario 1)*	3.9	3.9	3.9
h_{base} *(Scenario 2)*	5.8	5.8	5.8
h_{LT}	0.20	0.20	0.20
h_{RT}	-0.60	-0.60	-0.60
h_{HV}	1.70	1.70	1.70
P_{LT}	0.00	0.10	0.10
P_{RT}	0.10	0.00	0.00
P_{HV}	0.05	0.08	0.08
h_{adj}	0.025	0.156	0.156
h_s *(Scenario 1)*	3.925	4.056	4.056
h_s *(Scenario 2)*	5.825	5.956	5.956
X_j	0.267	0.384	0.343
P (Scenario 1)	0.733	0.616	0.657
P (Scenario 2)	0.267	0.384	0.343
P (DOC 1)	0.733	0.616	0.657
P (DOC 3)	0.267	0.384	0.343
AdjP (DOC 1)	0.005	0.012	0.013
AdjP (DOC 3)	-0.001	-0.012	-0.010
P' (Scenario 1)	0.739	0.628	0.670
P' (Scenario 2)	0.265	0.373	0.333
h_d	**4.4**	**4.8**	**4.7**

Westbound Solution

Iteration No.:	1	2	3
Demand Flow, NB	325	325	325
Demand Flow, EB	300	300	300
Initial h_d	**3.2**	**4.6**	**4.1**
h_{base} *(Scenario 1)*	3.9	3.9	3.9
h_{base} *(Scenario 2)*	5.8	5.8	5.8
h_{LT}	0.20	0.20	0.20
h_{RT}	-0.60	-0.60	-0.60
h_{HV}	1.70	1.70	1.70
P_{LT}	0.05	0.00	0.00
P_{RT}	0.00	0.05	0.05
P_{HV}	0.08	0.10	0.10
h_{adj}	0.146	0.140	0.140
h_s *(Scenario 1)*	4.046	4.040	4.040
h_s *(Scenario 2)*	5.946	5.940	5.940
X_j	0.289	0.401	0.430
P (Scenario 1)	0.711	0.599	0.570
P (Scenario 2)	0.289	0.401	0.430
P (DOC 1)	0.711	0.599	0.570
P (DOC 3)	0.289	0.401	0.430
AdjP (DOC 1)	0.006	0.008	0.009
AdjP (DOC 3)	-0.001	-0.120	-0.129
P' (Scenario 1)	0.717	0.607	0.578
P' (Scenario 2)	0.287	0.281	0.301
h_d	**4.6**	**4.1**	**4.1**

Step 5: Determine the Capacity of Controlled Approaches

The tables above show the results of the three iterations required to determine the departure headways for the two approaches. Now, the demand flow rate in each approach (separately) is incrementally increased – while keeping the demand on the conflicting approach constant. For each demand flow rate, a new set of iterations are needed to produce a departure headway. The demand flow rate on the subject approach is increased until the resulting degree of saturation (vh_d/3600) becomes 1.000. This is now an iteration of individual solutions, each of which is itself iterative. Obviously, we cannot show all of these computations. Suffice it to say that each iteration produces a table like the tables above, and iterations continue until the degree of saturation reaches 1.000.

For the sample problem, the following capacities are determined in this way:

c_{NB} = 764 veh/h
c_{EB} = 868 veh/h

Step 6: Determine Control Delay and LOS for Each Approach

Equation 25-27 is used to estimate the average control delay on each approach:

$$d_x = t_{sx} + 900T\left[(X_x - 1) + \sqrt{(X_x - 1)^2 + \frac{h_{dx}X_x}{450T}}\right] + 5$$

In each case, values are taken from the 3rd iteration of the solution shown in the previous tables. Remember that the service time, t_{sx}, for each case is the departure headway minus the move-up time, which has a default value of 2.0 s/veh for Geometry Group 1. Then:

t_{sNB}	=	4.5 – 2.0	=	2.5 s/veh
t_{sEB}	=	4.5 – 2.0	=	2.5 s/veh
T	=	0.25 h		
X_{NB}	=	325*4.7/3600=		0.424
X_{EB}	=	300*4.1/3600 =		0.342
h_{dNB}	=	4.7 s/veh		
h_{dEB}	=	4.1 s/veh		

and:

$$d_{NB} = 2.5 + 900 * 0.25\left[(0.424 - 1) + \sqrt{(0.424-1)^2 + \frac{4.7*0.424}{450*0.25}}\right] + 5 = 10.8\ s/veh$$

$$d_{EB} = 2.5 + 900 * 0.25\left[(0.342 - 1) + \sqrt{(0.342-1)^2 + \frac{4.1*0.342}{450*0.25}}\right] + 5 = 9.9\ s/veh$$

From Table 25-7, both approaches operate right near the LOS A/B boundary (10 s/veh). The NB approach is technically at LOS B, while the WB approach is at LOS A. Both approaches are operating very well under current demands.

Problem 25-3

The problem as stated is an analysis of a simple one-lane roundabout with all one-lane entry and exit roadways.

Steps 1 and 2: Convert Demand Volumes to Flow Rates in pc/h

These conversions are computed using Equations 25-1 and 25-28, which are combined for convenience:

$$v_i = \frac{V_i}{PHF * f_{HV}}$$

$$f_{HV} = \frac{1}{1+P_T}$$

The peak hour factor for all movements is 0.90 (given), and there are 5% trucks (0.05) in all volumes. Then:

$$f_{HV} = \frac{1}{1+0.05} = 0.952$$

$$PHF * f_{HV} = 0.90 * 0.952 = 0.857$$

$$v_i = V_i / 0.857$$

These computations are summarized in the table that follows:

Demand Flow Rates in pc/h

Approach	LT (pc/h)	TH (pc/h)	RT (pc/h)
NB	38/0.857 = 44	200/0.857 = 233	35/0.857 = 41
SB	35/0.857 = 41	220/0.857 = 257	45/0.857 = 53
EB	27/0.857 = 32	185/0.857 = 216	30/0.857 = 35
WB	27/0.857 = 32	175/0.857 = 204	20/0.857 = 23

Note: all flow rates rounded to the nearest whole number.

Step 3: Determine Circulating and Exiting Flow Rates
Table 25-16 shows equations for the computation of circulating and exiting flow rates at each approach to the roundabout. The table below summarizes these computations.

Approach	Circulating Flow Rate, v_c	Exiting Flow Rate, v_{ex}
NB	$v_{cNB} = v_{WBU} + v_{SBL} + v_{SBU} + v_{EBT} + v_{EBL} + v_{EBU}$ $v_{cNB} = 0 + 41 + 0 + 216 + 32 + 0 = 289$	$v_{exNB} = v_{EBR} + v_{SBT} + v_{WBL} + v_{NBU}$ $v_{exNB} = 35 + 257 + 32 + 0 = 324$
SB	$v_{cSB} = v_{EBU} + v_{NBL} + v_{NBU} + v_{WBT} + v_{WBL} + v_{WBU}$ $v_{cSB} = 0 + 44 + 0 + 204 + 32 + 0 = 280$	$v_{exSB} = v_{WBR} + v_{NBT} + v_{EBL} + v_{SBU}$ $v_{exSB} = 23 + 233 + 32 + 0 = 288$
EB	$v_{cEB} = v_{NBU} + v_{WBL} + v_{WBU} + v_{SBT} + v_{SBL} + v_{SBU}$ $v_{cEB} = 0 + 32 + 0 + 257 + 41 + 0 = 330$	$v_{exEB} = v_{SBR} + v_{WBT} + v_{NBL} + v_{EBU}$ $v_{exEB} = 53 + 204 + 44 + 0 = 301$
WB	$v_{cWB} = v_{SBU} + v_{EBL} + v_{EBU} + v_{NBT} + v_{NBL} + v_{NBU}$ $v_{cWB} = 0 + 32 + 0 + 233 + 44 + 0 = 309$	$v_{exWB} = v_{NBR} + v_{EBT} + v_{SBL} + v_{WBU}$ $v_{exWB} = 41 + 216 + 41 + 0 = 298$

Step 4: Determine Entry Flow Rates by Lane
As there is only one entry lane on each approach, this step is not necessary.

Step 5: Determine the Capacity of Each Entry and Bypass Lane
There are no bypass lanes in the proposed roundabout. The capacity of each entry lane is given by the equations in Table 25-17. As all of the entries are one-lane entries onto a one-lane roundabout, the following equation is used:

$$c = 1{,}130\, e^{-(0.001 v_c)}$$
$$c_{NB} = 1{,}130\, e^{-(0.001*289)} = 846\ pc/h$$
$$c_{SB} = 1{,}130\, e^{-(0.001*280)} = 854\ pc/h$$
$$c_{EB} = 1{,}130\, e^{-(0.001*330)} = 812\ pc/h$$
$$c_{SB} = 1{,}130\, e^{-(0.001*309)} = 830\ pc/h$$

As there are a negligible number of pedestrians and bicycles present, there are no adjustments needed to these values.

The capacity of exit roadways is roughly estimated (by the methodology) as 1,200 pc/h, which is well above any of the projected exit flow rates.

Step 6: Estimate the Control Delay and LOS for Each Entering Roadway
Equation 25-30 is used to estimate control delay in each entering lane – we have only one lane per approach. It uses the computed capacities of each entry lane, the total flow rate (in pc/h) on each entry lane, and the v/c ratio that results from these. These are summarized in the table below:

Approach	Flow Rate, v (pc/h)	Capacity, c (pc/h)	X = v/c
NB	44+233+41 = 318	846	0.376
SB	41+257+53 = 351	854	0.411
EB	32+216+35=283	812	0.349
WB	32+204+23 = 259	830	0.312

Then, using Equation 25-30:

$$d = \frac{3600}{c} + 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{\left(\frac{3600}{c}\right)X}{450T}}\right]+\left[5*\min(X,1)\right]$$

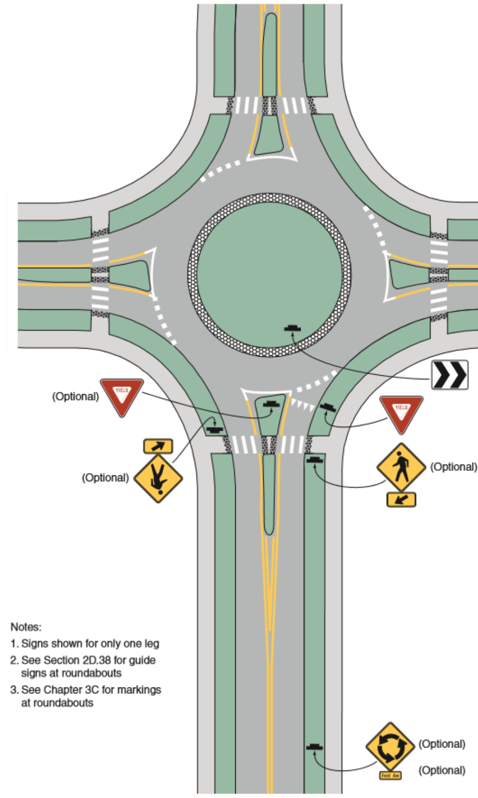
$$d_{NB} = \frac{3600}{846} + 900*0.25\left[(0.376-1)+\sqrt{(0.376-1)^2+\frac{\left(\frac{3600}{846}\right)0.376}{450*0.25}}\right]+\left[5*\min(0.376,1)\right] = 8.4\ s/veh$$

$$d_{SB} = \frac{3600}{854} + 900*0.25\left[(0.411-1)+\sqrt{(0.411-1)^2+\frac{\left(\frac{3600}{854}\right)0.411}{450*0.25}}\right]+\left[5*\min(0.411,1)\right] = 8.7\ s/veh$$

$$d_{EB} = \frac{3600}{812} + 900*0.25\left[(0.411-1)+\sqrt{(0.411-1)^2+\frac{\left(\frac{3600}{812}\right)0.411}{450*0.25}}\right]+\left[5*\min(0.411,1)\right] = 8.4\,s/veh$$

Note that the standard analysis period of 15 minutes (0.25h) is used. All of the delays are below 10 s/veh, which places all approaches in LOS A. The single-lane roundabout as proposed is expected to work extremely well under current demands.

The typical markings and signing for a one-lane roundabout is illustrated in the figure below, which is taken from the MUTCD:



Solutions to Problems in Chapter 26

Interchanges and Alternative Intersections

Problem 26-1

In general, the interchange form that consumes the least land is the single-point urban interchanges, or SPUI. It is particularly well-suited to a situation with a high degree of left turns exiting the freeway. It also handles heavy left turning movements onto the freeway, although those are not mentioned in the problem statement. The most significant benefit of this type of interchange is that connections with the surface street are handled through a single signalized intersection.

Problem 26-2

A split diamond interchange with two one-way arterials creates four intersections, all of which may or may not be signalized. With a split diamond interchange the number and complexity of movements at each of the intersections is reduced when compared to the standard diamond interchange with a two-way arterial. At each intersection, the following movements exist (split diamond):

- Through movement from or to a ramp.
- A left turn *or* right turn from the ramp to the arterial.
- Through movement on the arterial.
- A left turn *or* right turn from the arterial to the ramp.

In the split diamond, all of the left turns would be unopposed, coming from one-way roadways (either the ramp or the arterial).

In a standard diamond interchange, each of the *two* intersections formed would be a one-way ramp with left and right turns, and a two-way arterial with left, through, and/or right turn movements in each direction.

Problem 26-3

The question asks for the determination of the level of service for Movements O_1 to D_4 and O_1 to D_3. The level of service is based upon the parameter "experienced travel time," or ETT. It is computed as:

$$ETT = \sum d_i + \sum EDTT$$

Movement O_1 to D_4

For Movement O_1 to D_4, the path through the interchange is 12 – 10 – 6 – 5 – 4 – 3. If a typical 4-leg surface intersection were present, the path would be 12 – 9 – 5 – 4 – 3. A portion of this path is the same: the time taken to travel the distance 5 – 4 – 3 may be ignored. The "extra distance travel time," (*EDTT*) is the difference between the interchange path 12 – 10 – 6 – 5, and the hypothetical alternative for a surface intersection, 12 – 9 – 5.

Interchange Path
With the interchange path, vehicles must go through two signalized intersections at points 10 and 6. The delay experienced at each is 24.6 s (given). Therefore:

$$\sum d_i = 24.6 + 24.6 = 49.2 \ s/veh$$

The travel time for each segment of the interchange path is as follows, given that ramp speeds are 30 mi/h, and arterial speeds are 45 mi/h. It is assumed that the connecting link 10 – 6 has a 30 mi/h speed. Then:

- TT (12-10) = 1,000/1.47*30 = 22.7 s/veh
- TT (10-6) = 1,200/1.47*30 = 27.2 s/veh
- TT (6-5) = 200/1.47*45 = 3.0 s/veh
- Total TT = = **52.9 s/veh**

Note that distances 5 – 4 – 3 are ignored, because they are same in the hypothetical alternative.

Surface Intersection Path
If this were a surface intersection, the OD path would be 12 – 9 – 5, again ignoring distances 5 – 4 – 3. Assuming that the freeway was replaced with a surface arterial with a free-flow speed of 45 mi/h, the travel time would be:

- TT (12 – 9) = 980/1.47*45 = 14.8 s/veh
- TT (9 – 5) = 1,200/1.47*45 = 18.1 s/veh
- Total TT = = **32.9 s/veh**

EDTT
The extra-distance travel time for this movement is 52.9 – 32.9 = 20.0 s/veh.

ETT and LOS
The experienced travel time for this movement is 49.2 + 20.0 = 69.2 s/veh.

The level of service for this movement is D, from Table 26-1.

Movement O_1 to D_3
Because this is essentially a right turn movement, it is much simpler than other movements. The interchange path is 12 – 10. The hypothetical path for a surface intersection is 12 – 5 – 10.

Interchange Path
The OD movement goes through one signalized intersection. Therefore:

$$\sum d_i = 24.6 \ s$$

The travel time for link 12 – 10 is 1,000/1.47*30 = **22.7 s/veh**

Surface Intersection Path
The travel time for segments of the hypothetical intersection path is:

- TT (12-9) = 980/1,47*45 = 14.8 s
- TT (9-10) = 200/1.47*45 = 3.0 s
- Total = = **17.8 s**

EDTT
The extra distance travel time in this case is 22.7 – 17.8 = 4.9 s.

Note that in this case, while the physical distance in the interchange is less than that on the hypothetical intersection, the slower speeds prevalent on the ramp create a positive *EDTT.*

ETT and LOS
The *ETT* for this movement is 22.7 + 4.9 = 27.6 s/veh. This is LOS B from Table 26-1.

NOTE: Neither movement includes the delay at the hypothetical signalized intersection of two arterials, or its comparison to the sum of the delays of the individual intersections through which the movements follow. This may be considered a flaw in the methodology.

<u>Problem 26-4</u>
Given the design of the RCUT intersection, both through movements and left-turn movements from First Street will have the same characteristics and level of service. Therefore, only one set is examined for this problem.

Through Movement
Through vehicles on First Street must turn right onto Main Road, execute a U-Turn using the provided lanes, return to First Street and execute another right turn, back onto First Street. The hypothetical alternative path through a standard intersection would be straight across Main Road.

In making this maneuver, through vehicles go through two signals, one in each direction at the primary intersection. They turn right from First Street, experiencing 22.0 s/veh of delay, and then right from Main Road, experiencing 15.5 s/veh (after executing the U-turn). Thus:

$$\sum d_i = 12.0 + 11.5 = 23.5\, s/veh$$

The time spent getting to the U-turn lane, making the U-turn, and returning to First Street is:

$$\frac{425}{1.47*45} + 8.2 + \frac{425}{1.47*45} = 4.9 + 8.2 + 4.9 = 18.0\ s$$

In the hypothetical case of a standard intersection, through vehicles would simply cross Main Road (a distance of 112 ft) at a speed of 30 mi/h. This maneuver would take:

$$\frac{112}{1.47*30}=2.5\ s$$

Thus, for the through movement on First Street:

$$EDTT=18.0-2.5=15.5\ s/veh$$

and:

$$ETT=23.5+15.5=39.0\ s$$

From Table 26-1, this is LOS D.

Left-Turn Movement
The left turn movement is similar to the through movement, except that vehicles travel through the signal at First Street after their U-Turn.

The delay these vehicles experience at the two intersections is:

$$\sum d_i=12.0+10.0=22.0\ s/veh$$

The travel time experienced by left-turners as they get to the U-Turn, make the U-Turn, and return to the primary intersection is the same as that for through vehicles:15.5 s/veh. This assumes that the hypothetical time to traverse a standard intersection would be similar to that for through vehicles (this is a simplification).

Then:

$$ETT=22.0+15.5=37.5\ s/veh$$

From Table 26-1, this is LOS D.

For First Street vehicles, the LOS is not great. An overall evaluation of the intersection, however, would have to look at delays to Main Road vehicles under the RCUT case, with simpler signal timing (and most likely a shorter cycle length), and the more complex multiphase signal timing that would have to exist to signalize a standard intersection design.

Problem 26-5
An external approach to a PARCLO interchange with two signalized intersections has the following characteristics:

- Demand Flow Rate = 1,300 veh/h.
- 2-lane arterial approach.
- Distance between two intersections = 850 ft.
- At the second intersection, 25% of the approach flow will turn right, 57% will go through, and 18% will turn right.

The lane utilization factor is computed as:

$$P_{Li} = \frac{1}{N} + a_1\left(\frac{v_R}{v_L + v_T + v_R}\right) + a_2\left(\frac{v_L}{v_L + v_T + v_R}\right) + a_3\left(\frac{D * v_L}{10^6}\right)$$

where:

N	=	2 (given)		
v_R	=	1,300*0.00	=	0 veh/h
v_T	=	1,300*0.60	=	780 veh/h
v_L	=	1,300*0.40	=	520 veh/h
D	=	850 ft (given)		
a_1	=	0.387 (Table 26-2, PARCLO B-2Q, 2 lanes)		
a_2	=	-0.344 (Table 26-2, PARCLO B-2Q, 2 lanes)		
a_3	=	0.000 (Table 26-2, PARCLO B-2Q, 2 lanes)		

Note that the coefficients a_i are for the leftmost lane. No values are provided for the rightmost lane. Then:

$$P_{L2} = \frac{1}{2} + 0.387\left(\frac{0}{520 + 780 + 0}\right) - 0.344\left(\frac{520}{520 + 780 + 0}\right) + 0.00\left(\frac{850 * 520}{10^6}\right) = 0.362$$

Thus, the leftmost lane will carry 0.362*1,300 = 471 veh/h, and the rightmost lane will carry 1,300 – 471 or 829 veh/h. Thus, the value of P_{Lmax} is 829/1300 = 0.638.

The lane utilization factor is given by Equation 26-5:

$$f_{LU} = \frac{1}{P_{L\max} N} = \frac{1}{0.638 * 2} = 0.784$$

<u>Problem 26-6</u>

A DDI has three lanes approaching the external crossover, which is signalized. At the internal crossover, the three lanes consist of one exclusive LT lane, and two through lanes. The lane utilization factor for the external crossover approach is to be determined.

Lane distribution at the external crossover intersection is defined by Equation 34-8:

$$P_{LiDDI} = a_1\, LTDR + a_2$$

where $LTDR$ =1,200/2,800 = 0.429. Values of coefficients a_1 and a_2 are selected from Figure 22-17 for the 3-Lane Exclusive case. The Regime is II, as $LTDR > 0.333$. Then:

Coefficient	**Leftmost Lane**
a_1	0.9695
a_2	0.0096

and:

$$P_{L3DDI} = 0.9695 * 0.429 + 0.0096 = 0.426$$

The leftmost lane is considered to be the peak lane in this case, and:

$$f_{LUDDI} = \frac{1}{P_{L\max DDI} * N} = \frac{1}{0.426 * 3} = 0.782$$

<u>Problem 26-7</u>

For an exclusive LT lane, the left-turn adjustment factor is equal to the factor for turning radius given in Equation 26-9:

$$f_{LT} = f_R = \frac{1}{1+\left(\frac{5.61}{R}\right)} = \frac{1}{1+\left(\frac{5.61}{250}\right)} = 0.978$$

Solutions to Problems in Chapter 27

Overview of the Geometric Design of Roadways

Problem 27-1

Information: Horizontal curve, P.I. = 11,500 + 66
Radius = 1,000 ft
Angle of deflection = 60°

Find: All relevant characteristics of the curve. Stations of the P.C. and P.T.

The degree of curvature is computed using Equation 27-1:

$$D = \frac{5{,}729.58}{R} = \frac{5{,}729.58}{1{,}000} = 5.73^{o}$$

Equations for key characteristics of the defined circular curve are found in Table 27-1. Then:

$$T = R\ Tan\left(\Delta/2\right) = 1{,}000\ Tan\left(60/2\right) = 1{,}000\ Tan(30) = 1{,}000 * 0.5773 = 577.3\ \ ft$$

$$L = 100\left(\frac{\Delta}{D}\right) = 100\left(\frac{60}{5.73}\right) = 100 * 10.47 = 1{,}047\ \ ft$$

$$M = R\left[1 - Cos\left(\Delta/2\right)\right] = 1{,}000 * \left[1 - Cos\left(60/2\right)\right] = 1{,}000 * 0.1340 = 134\ \ ft$$

$$E = R\left[\left(\frac{1}{Cos\left(\Delta/2\right)}\right) - 1\right] = 1{,}000\left[\left(\frac{1}{Cos(30)}\right) - 1\right] = 1{,}000 * \left[0.1547\right] = 154.7\ \ ft$$

$$LC = 2R\,Sin\left(\Delta/2\right) = 2 * 1{,}000 * Sin(30) = 2{,}000 * 0.5000 = 1{,}000\ ft$$

The stationing then becomes:

P.C. = (11,500+66)-577.3=10,988.7 = 11,900+88.7
P.T. = (11,900+88.7)+1,047=13,035.7=13,000+35.7

Problem 27-2

Information: 3.5° curve
60 mi/h design speed
Two 12-ft lanes
Use spiral transition curves
P.I. = 15,100+26
Angle of deflection =40°

Find: T.S., S.C., C.S., and S.T.

The design value of superelevation is given by Equation 27-3:

$$e = 100\left[\left(\frac{S_{des}^2}{15R}\right) - f_{des}\right]$$

From Table 27-2, the design friction factor (f_{des}) = 0.12. Then:

$$R = \frac{5{,}729.58}{D} = \frac{5{,}729.58}{3.5} = 1{,}637\ \ ft$$
$$e = 100\left[\left(\frac{60^2}{15*1{,}637}\right) - 0.12\right] = 2.6\%$$

The length of the spiral is typically controlled by the minimum and maximum values, the equations for which are found in Table 27-5, and the AASHTO-recommended value, which is found in Table 27-6:

$$L_{SP\min} = \sqrt{15.84R} = \sqrt{15.84*1{,}637} = \sqrt{25{,}930} = 161\ \ ft$$
$$or$$
$$L_{SP\min} = 0.7875\frac{S^3}{R} = 0.7875\frac{60^3}{1{,}637} = 104\ \ ft$$
$$L_{SP\min} = 104\ \ ft$$
$$L_{SP\max} = \sqrt{79.2\,R} = \sqrt{79.2*1{,}637} = \sqrt{129{,}650.4} = 360\ \ ft$$
$$L_{SP\,rec} = 176\ \ ft\ (Table\ 27-6)$$

It may also be computed as the length of the spiral runoff, or the length of the spiral plus tangent runoffs. These are estimated using Equations 27-4 and 27-5, using the following assumptions: Superelevation is achieved by rotating both lanes around the centerline, and the normal drainage cross-slope is 1%. Then:

$$L_S = \frac{wne_d\ b_w}{\Delta}$$
$$L_t = \frac{e_{NC}}{e_d}L_r$$

Where:	w	=	12 ft
	n	=	1 lanes rotated around centerline
	e_{NC}	=	1%
	e_d	=	2.6%
	b_w	=	1.00 (Table 27-3)
	Δ	=	0.45 (Table 27-4, 60 mi/h)

Then:

$$L_S = \frac{12*1*2.6*1.00}{0.45} = 69.3 \ ft$$
$$L_t = \left(\frac{1}{2.6}\right)*69.3 = 26.7 \ ft$$
$$L_S + L_t = 69.3 + 26.7 = 96.0 \ ft$$

The recommended value of 176 ft lies between the minimum and maximum allowable values, and will be used in this case.

The central angle for the spiral is given by: (Table 27-5)

$$\delta = \frac{L_S \, D}{200} = \frac{176*3.5}{200} = 3.1^o$$

The central angle of deflection for the circular portion of the curve is given by: (Table 27-5)

$$\Delta_S = \Delta - 2\delta = 40.0 - (2*3.1) = 33.8^o$$

Then:

$$T_S = \left[R\,Tan\left(\frac{\Delta}{2}\right)\right] + \left[\left(R\,Cos(\delta) - R + \frac{L_{SP}^2}{6R}\right)*Tan\left(\frac{\Delta}{2}\right)\right] + \left[L_{SP} - R\,Sin(\delta)\right]$$
$$T_S = \left[1637\,Tan\left(\frac{40}{2}\right)\right] + \left[\left(1637\,Cos(3.1) - 1637 + \left(\frac{176^2}{6*1637}\right)\right)*\left(Tan\left(\frac{40}{2}\right)\right)\right] + \left[176 - 1637\,Sin(3.1)\right]$$
$$T_S = [1637*0.3639] + [((1637*0.9985) - 1637 + 3.2)*0.3639] + [176 - (1637*0.0541)]$$
$$T_S = 595.7 + 0.7445 + 87.4 = 683.8 \ ft$$

and:

$$L_c = 100\left(\frac{\Delta_s}{D}\right) = 100\left(\frac{33.8}{3.5}\right) = 965.7 \ ft$$

Then:

$$T.S. = P.I. - T_s = (15{,}100 + 26) - 683.8 = 14{,}442.2 = 14{,}400 + 42.2$$
$$S.C. = T.S. + L_{SP} = 14{,}442.2 + 176 = 14{,}618.2 = 14{,}600 + 18.2$$
$$C.S. = S.C. + L_c = 14{,}618.2 + 965.7 = 15{,}583.9 = 15{,}500 + 83.9$$
$$S.T. = C.S. + L_{SP} = 15{,}583.9 + 176 = 15{,}760 = 15{,}700 + 60.0$$

Problem 27-3

Information: 5^o curve
65 mi/h design speed
2% upgrade
t = 2.5 s (driver reaction time)

Find: Closest placement of roadside object.

Placement of roadside objects is based upon the severity of the curve and the safe-stopping distance, computed as:

$$d_s = 1.47\,S\,t + \frac{S^2}{30(0.348+G)} = (1.47*65*2.5) + \left(\frac{65^2}{30*(0.348+0.02)}\right)$$
$$d_s = 238.9 + 382.7 = 621.6 \ \ ft$$

Then: (Equation 27-6)

$$M = \frac{5729.58}{D}\left[1 - Cos\left(\frac{d_s D}{200}\right)\right] = \frac{5729.58}{5}\left[1 - Cos\left(\frac{621.6*5}{200}\right)\right]$$
$$M = 1{,}145.9*\left[1 - Cos(15.54)\right] = 1{,}145.9*0.036551 = 41.9 \ \ ft$$

Problem 27-4

Information: R = 1,200 ft
60 mi/h design speed
6% maximum superelevation rate

Find: Appropriate superelevation rate

Using Equation 27-3:

$$e = \left[\left(\frac{S_{des}^2}{15R}\right) - f_{des}\right]$$

Where: f_{des} = 0.12 (Table 27-2, 60 mi/h)

$$e = \left[\left(\frac{60^2}{15*1200}\right) - 0.12\right] = 0.08 \ or \ 8\%$$

As the resulting superelevation rate is *higher* than the maximum, this implies that a radius as short as 1,200 ft cannot be used. Equation 27-2 would be used to compute the *minimum* radius that could be used in this case:

$$R_{\min} = \frac{S^2}{15(e+f)} = \frac{60^2}{15(0.06+0.12)} = 1{,}333 \ \ ft$$

The proposed curve would have to be re-designed with a radius of *at least* 1,333 ft.

Problem 27-5

Information: e = 10%
70 mi/h design speed
Three 12-ft lanes
Rotation around pavement edge

Find: Length of superelevation runoff

Using Equation 27-4:

$$L_S = \frac{wne_d b_w}{\Delta}$$

Where: w = 12 ft (lane width)
N = 3 (lanes rotated)
e_d = 10% (given)
b_w = 0.67 (Table 27-4, 3 lanes rotated)
Δ = 0.40 (Table 27-3, 70 mi/h)

$$L_{SP} = \frac{12*3*10*0.67}{0.40} = 603 \ ft$$

Problem 27-6

Information:

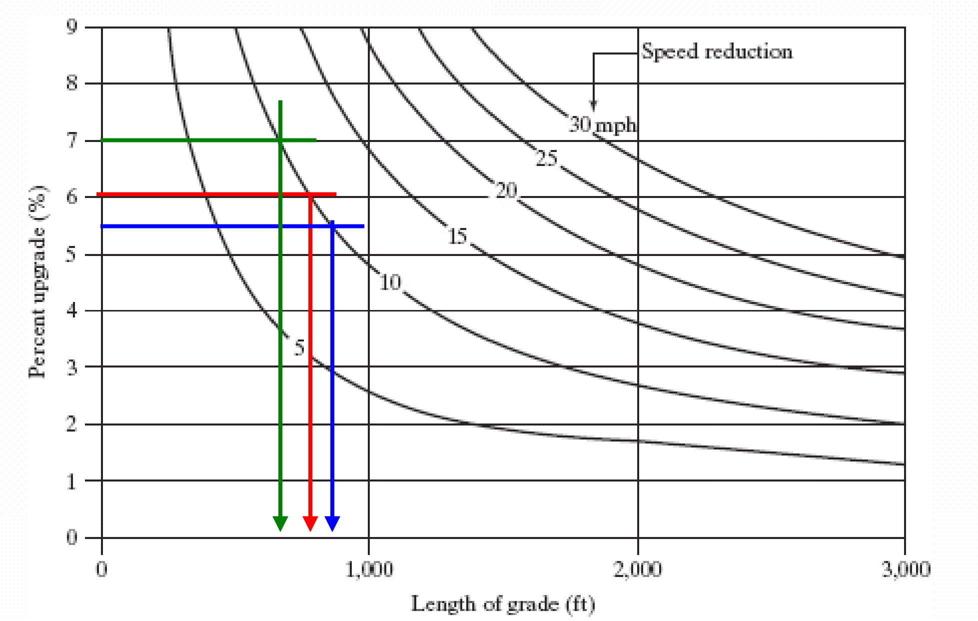
Category	Grade 1	Grade 2	Grade 3
Facility Type	Rural Freeway	Rural Arterial	Urban Arterial
Terrain	Mountainous	Rolling	Level
Design Speed	60 mi/h	45 mi/h	40 mi/h

Find: Maximum grade and critical length of grade for each

From Table 27-7:

Maximum grades are: Grade 1 = 6%
Grade 2 = 5.5%
Grade 3 = 7%

Critical lengths of grades are found from Figure 27-12, as shown below. Because all design speeds are less than 70 mi/h, a 10% reduction in speeds is used to determine the critical length.



Grade 1:	800 ft
Grade 2:	850 ft
Grade 3:	700 ft

<u>Problem 27-7</u>

Information: Vertical curve
+4% grade to -5% grade
VPI = 1,500+55
Elevation of VPI = 500 ft
L = 1,000 ft

Find: Stations of the VPC and VPT.
Elevations of the VPC and VPT.
Elevation points along the curve at 100-ft intervals
Location and elevation of the high point

Note that the curve described is a *crest vertical curve*.

An equation for this vertical curve can be constructed in the form of:

$$Y_x = ax^2 + bx + Y_o$$

Where:

$$a = \frac{G_2 - G_1}{200L} = \frac{-5-4}{200*1000} = -0.000045$$

$$b = \frac{G_1}{100} = \frac{4}{100} = 0.04$$

$$L = 1000 \ ft$$

$$Y_o = Y_{VPC} = 500.00 - (0.04*500) = 480.0 \ ft$$

Thus:

$$Y_x = -0.000045x^2 + 0.04x + 480.0$$

Stations: VPC = (1500+55)-500 = 1,000+55
VPT = (1500+55)+500 = 2,000+55

Elevations:

$$Y_x = -0.000045x^2 + 0.04x + 480.0$$
$$Y_o = Y_{VPC} = 480\ \ ft$$
$$Y_{100} = -0.000045(100^2) + (0.04*100) + 480 = 483.6\ \ ft$$
$$Y_2 = -0.000045(200^2) + (0.04*200) + 480 = 486.2\ \ ft$$
$$Y_3 = -0.000045(300^2) + (0.04*300) + 480 = 488.0\ \ ft$$
$$Y_4 = -0.000045(400^2) + (0.04*400) + 480 = 488.8\ \ ft$$
$$Y_5 = -0.000045(500^2) + (0.04*500) + 480 = 488.8\ \ ft$$
$$Y_6 = Y_{PVT} = -0.0045(600^2) + (0.04*600) + 480 = 487.9\ \ ft$$

The high point is found as:

$$x = \frac{-G_1L}{G_2 - G_1} = \frac{-4*1000}{-5-4} = \frac{-4000}{-9} = 444\ \ ft$$
$$Y_{4.44} = -0.000045(444^2) + (0.04*444) + 480 = 488.9\ \ ft$$

<u>Problem 27-8</u>
Information:

Grade	Entry Grade	Exit Grade	Design Speed	Reaction Time
1	3%	8%	45 mi/h	2.5 s
2	-4%	2%	65 mi/h	2.5 s
3	0%	-3%	70 mi/h	2.5 s

Find: Minimum lengths of the above vertical curves.

To find the minimum lengths of grade, the safe stopping distance for each curve must be computed. To do this, the grade used in the computation will be grade which results in the worst (or highest) safe stopping distance.

$$d_s = 1.47\,S\,t + \frac{S^2}{30(0.348+G)}$$

$$d_{s1} = (1.47*45*2.5) + \left[\frac{45^2}{30*(0.348+0.03)}\right] = 165.4 + 178.6 = 344.0\ ft$$

$$d_{s2} = (1.47*65*2.5) + \left[\frac{65^2}{30*(0.348-0.04)}\right] = 238.9 + 457.3 = 696.2\ ft$$

$$d_{s3} = (1.47*70*2.5) + \left[\frac{70^2}{30*(0.348-0.03)}\right] = 257.3 + 513.6 = 770.9\ ft$$

It should be noted that Curve 1 is a *SAG* vertical curve; Curve 2 is a *SAG* vertical curve; Curve 3 is a *CREST* vertical curve.

We will start each computation assuming that the length of the curve is *greater* than the safe stopping distance. Equations are from Table 27-8:

<u>Curve 1</u>

$$L = \frac{A d_s^2}{400+3.5d_s} = \frac{|8-3|*344^2}{400+(3.5*344)} = \frac{591{,}680}{1{,}604} = 368.9\ ft > 344\ ft \quad OK$$

<u>Curve 2</u>

$$L = \frac{A d_s^2}{400+3.5d_s} = \frac{|-2-(-4)|*696.2^2}{400+(3.5*696.2)} = \frac{2{,}908{,}166.6}{2{,}836.7} = 1{,}025.2\ ft > 696.2\ ft\ \ OK$$

<u>Curve 3</u>

$$L = \frac{A d_s^2}{2{,}158} = \frac{|0-(-3)|*770.9^2}{2{,}158} = \frac{1{,}782{,}860}{2{,}158} = 826.2\ ft > 770.9\ ft\ \ OK$$

<u>Problem 27-9</u>

Information:
- Vertical curve
- -4% to +1%
- Minimum length curve
- t = 2.5 s
- 70 mi/h design speed
- VPI = 5100+22
- Elevation of the VPI = 1,285 ft

Find:
- VPC and VPT
- Elevation of points on 100-ft intervals
- High point and station

To begin, we must determine the minimum length of curve. Note that this is a *SAG* vertical curve.

$$d_s = 1.47\,S\,t + \left(\frac{S^2}{30*(0.348+G} \right)$$

$$d_s = (1.47*70*2.5) + \left[\frac{70^2}{30*(0.348-.04)} \right] = 257.3 + 530.3 = 787.6\ ft$$

Assuming that L > d_s: (Table 27-8)

$$L = \frac{Ad_s^2}{400+3.5d_s} = \frac{|-4-1|*787.6^2}{400+(3.5*787.6)} = \frac{3{,}101{,}568.8}{3{,}156.6} = 982.6\ ft > 787.6\ ft \quad OK$$

For convenience of construction, we will round off to *L* = 985 ft

Then:

$$a = \frac{G_2 - G_1}{200L} = \frac{-4-1}{200*985} = -0.0000254$$

$$b = \frac{G_1}{100} = \frac{-4}{100} = -0.04$$

$$Y_o = Y_{VPC} = 1285 + 0.04(985/2) = 1{,}304.7$$

$$Y_x = -0.0000254x^2 - 4x + 1{,}304.7$$

Elevations are now computed for intervals of x = 100 ft from “0” to 985 ft (which is the end of the curve). The station of the VPC is (5100+22)-(985/2)=4600+29.4. The station of the VPT is (5100+22)+(985/2) = 5,600+14.6. The spreadsheet table below shows the resulting elevations:

L	*Y*
0	1304.7
100	1300.4
200	1295.7
300	1290.4
400	1284.6
500	1278.4
600	1271.6
700	1264.3
800	1256.4
900	1248.1
985	1240.7

Because of the two grades involved, the high point of this curve is at its beginning, or 1,304.7 ft.

Solutions to Problems in Chapter 28

Capacity and Level of Service Analysis: Basic Freeway and Multilane Highway Segments

Problem 28-1

The free-flow speed of a multilane highway is estimated using Equation 28-3:

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A$$

Where:	BFFS	=	60 mi/h (given)
	f_{LW}	=	1.9 mi/h (Table 28-3, 11-ft lanes)
	f_{LC}	=	0.65 mi/h (Table 28-5, 3 + 6 = 9 ft total lateral clearance, interpolated)
	f_M	=	1.6 mi/h (Table 28-6, undivided)
	f_A	=	3.75 mi/h (Table 28-7, 15 access pts/mi, interpolated)

$$FFS = 60.00 - 1.90 - 0.65 - 3.75 = 53.7\ mi/h$$

Problem 28-2

The free-flow speed of a freeway is estimated using Equation 28-2:

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22 TRD^{0.84}$$

Where:	f_{LW}	=	0.0 mi/h (Table 28-3, 12-ft lanes)
	f_{LC}	=	1.6 mi/h (Table 28-4, 2-ft clearance, 3-lanes)
	TRD	=	3.5 ramps/mi (given)

$$FFS = 75.4 - 0.0 - 1.6 - 3.22(3.5^{0.84}) = 75.4 - 0.0 - 1.6 - 9.2 = 64.6\ mi/h$$

Problem 28-3

The average grade is based on the total rise divided by the horizontal length over which the rise takes place:

Rise on 2% grade = 1,000*0.02 =	20 ft
Rise on 3% grade = 1,500*0.03 =	45 ft
Total Rise =	*65 ft*

$$G_{AV} = \frac{65}{1{,}000 + 1{,}500} * 100 = 2.6\%$$

Problem 28-4

From Table 28-17, for rolling terrain, E_{HV} = 3.0. Then:

PC Equivalents for Trucks:	3,200*0.15*3.0	=	1,440 pc/h
PC Equivalents for Cars:	3,200*0.85*1.0	=	2,720 pc/h
Total Equivalent Volume:			**4,160 pc/h**

Problem 28-5

The free-flow speed of the subject freeway is determined using Eqn 28-2:

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22\ TRD^{0.84}$$

where: f_{LW} = 1.9 mi/h (Table 28-3, 11-ft lanes)
f_{LC} = 0.6 mi/h (Table 28-4, 3-ft clearance, 4 lanes)
TRD = 4.2 ramps/mi (given)

$$FFS = 75.4 - 1.9 - 0.6 - 3.22\left(4.2^{0.84}\right) = 61.8\ mi/h$$

Service flow rates are computed using Eqn 28-13; service volumes are computed using Eqn 28-14:

$$SF = MSF * N * f_{HV} * f_p$$
$$SV = SF * PHF$$

Maximum service flow rates (MSF) are selected from Table 28-15 for a FFS of 61.8 mi/h by interpolation:

- MSF_A = 678 pc/h/ln
- MSF_B = 1,112 pc/h/ln
- MSF_C = 1,585 pc/h/ln
- MSF_D = 2,017 pc/h/ln
- MSF_E = 2,318 pc/h/ln

These could have also been determined by plotting the calibrated speed-flow curve for the segment, and constructing density boundary lines. The scale of such graphics, however, make interpolation in Table 28-15 a more accurate approach.

The heavy vehicle factor is based upon passenger car equivalents for trucks on a 3.5% grade of 1.5 miles. The pce values are different for the upgrade and the downgrade.

E_T (upgrade) = 6.050 (Table 28-18, 3.5% grade, 1.5 mi, 3% trucks, interpolated)
E_T (dngrade) = 2.285 (Table 28-18, <0% grade, 3% trucks, interpolated)

Then, using Eqn 28-16:

$$f_{HV} = \frac{1}{1 + P_{HV}(E_{HV} - 1)}$$

$$f_{HV}(upgrade) = \frac{1}{1 + 0.03(6.05 - 1)} = 0.868$$

$$f_{HV}(dngrade) = \frac{1}{1 + 0.03(2.285 - 1)} = 0.963$$

The PHF is given as 0.92, there are 4 lanes in each direction on the freeway. Equations 28-13 and 28-14 are implemented in the spreadsheet table shown below.

LOS	MSF	N	f_{HV}	SF	PHF	SV
UPGRADE						
A	678	4	0.868	2354	0.92	2166
B	1112	4	0.868	3861	0.92	3552
C	1585	4	0.868	5503	0.92	5063
D	2017	**4**	0.868	7003	0.92	6443
E	2318	4	0.868	8048	0.92	7404
DOWNGRADE						
A	678	4	0.963	2612	0.92	2403
B	1112	4	0.963	4283	0.92	3941
C	1585	4	0.963	6105	0.92	5617
D	2017	4	0.963	7769	0.92	7148

Note that service flow rates and service volumes are in units of veh/h.

<u>Problem 28-6</u>

To determine the probable LOS for this existing 6-lane multilane highway with a measured FFS of 45 mi/h, the equivalent ideal lane flow must be determined using Eqn 28-11:

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

Where:

	V	=	4,000 veh/h (given)
	PHF	=	0.88 (given)
	N	=	3 lanes (given)
	E_{HV}	=	3.0 (Table 28-17, Rolling Terrain)

Then:

$$f_{HV} = \frac{1}{1+0.10(3.0-1)} = 0.833$$
$$v_p = \frac{4,000}{0.88*3*0.833} = 1,601 \ pc/h/\ln$$

Note that because this is a general terrain segment, the mix of trucks does not affect the value of E_{HV}. Also, because service flow rates for a 45-mi/h multilane highway are shown in Table 28-15, there is no need to calibrate a specific equation to determine these values. Comparing 1,601 pc/h/ln to the criteria in Table 28-15, it is seen that the expected LOS is E.

Problem 28-7

This is a design application for a section of freeway that goes from level terrain to a sustained 5%, 2-mile grade. LOS C is the design target. The number of lanes needed to provide this on the (a) upgrade, (b) downgrade, and (c) level terrain is needed. Equation 28-12 is used:

$$N = \frac{DDHV}{PHF * MSF * f_{HV}}$$

The FFS of the facility must be estimated to begin:

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22TRD^{0.84}$$
$$FFS = 75.4 - 0 - 0 - 3.22(1.0^{0.84}) = 75.4 - 3.22 = 72.2 \ mi/h$$

DDHV	=	2,500 veh/h (given),
MSF_C	=	1,954 pc/h/ln (Table 28-8, LOS C, interpolated between 70 mi/h and 75 mi/h), and
PHF	=	0.92 (given).

There may be as many as three different heavy vehicle adjustment factors for the three segments to be analyzed. They are based upon the appropriate passenger car equivalents for trucks and RVs. Level terrain values are selected from Table 28-17; upgrade (4.5%, 2 mi) values are selected from Table 28-18; downgrade values are also selected from Table 28-18 (<0%, 2 mi). The resulting values are shown below:

- E_{HV} = 2.00 (level terrain)
- E_{HV} = 2.87 (4.5%, 2 mi, 15% trucks)
- E_{HV} = 1.89 (<0%, 2 mi, 15% trucks)

Then:

$$f_{HV}\,(level) = \frac{1}{1+0.15(2.0-1)} = 0.870$$

$$f_{HV}\,(upgrade) = \frac{1}{1+0.15(2.87-1)} = 0.781$$

$$f_{HV}\,(downgrade) = \frac{1}{1+0.15(1.89-1)} = 0.882$$

Then:

$$N_{level} = \frac{2{,}500}{0.92*1{,}954*0.870} = 1.60\ lanes,\ SAY\ 2\ lanes$$

$$N_{upgrade} = \frac{2{,}500}{0.92*1{,}954*0.781} = 1.78\ lanes,\ SAY\ 2\ lanes$$

$$N_{downgrade} = \frac{2{,}500}{0.92*1{,}954*0.882} = 1.58\ lanes,\ SAY\ 2\ lanes$$

It appears that the provision of a 4-lane freeway will be sufficient to deliver LOS C on all of the defined segments.

<u>Problem 28-8</u>

This question concerns an old freeway with projected traffic growth in the future. It asks for an evaluation of LOS at various future time-points. The easiest way to approach this problem is to create a table of service volumes for the freeway which can be matched against future demand levels. To do this, we have to estimate the FFS of the freeway using Eqn 28-2:

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22\,TRD^{0.84}$$

where:	f_{LW}	=	1.9 mi/h (Table 28-3, 11-ft lanes)
	f_{LC}	=	3.6 mi/h (Table 28-4, 0-ft clearance, 2 lanes)
	TRD	=	4.5 ramps/mi (given)

$$FFS = 75.4 - 1.9 - 3.6 - 3.22(4.5^{084}) = 58.5\ mi/h$$

Then, using Equations 28-13 and 28-14:

$$SF = MSF * N * f_{HV} * f_p$$

$$SV = SF * PHF$$

where:	N	=	2 lanes (given)
	PHF	=	0.90 (given)

E_{HV} = 3.0 (Table 28-17, rolling terrain)
f_{HV} = 1/[1+0.05(3.0-1)] = 0.909

and values of MSF are selected for each LOS from Table 28-15 for a FFS or 58.5 mi/h, by interpolation:

- MSF_A = 642 pc/h/ln
- MSF_B = 1053 pc/h/ln
- MSF_C = 1522 pc/h/ln
- MSF_D = 2026 pc/h/ln
- MSF_E = 2285 pc/h/ln.

Equations 28-13 and 28-14 are implemented in the spreadsheet table shown below:

LOS	MSF	N	f_{HV}	SF	PHF	SV
A	642	2	0.909	1167	0.92	1074
B	1053	2	0.909	1914	0.92	1761
C	1522	2	0.909	2767	0.92	2546
D	2026	**2**	0.909	3683	0.92	3389

These values must be compared to the projected demand volumes over the next 20 years to determine the likely LOS that will exist:

Current Volume	=	2,100	=	2,100 veh/h (LOS C)
5-Year Forecast	=	$2{,}100 * 1.03^{5}$	=	2,434 veh/h (LOS C)
10-Year Forecast	=	$2{,}100 * 1.03^{10}$	=	2,822 veh/h (LOS D)
15-Year Forecast	=	$2{,}100 * 1.03^{15}$	=	3,271 veh/h (LOS D)
20-Year Forecast	=	$2{,}100 * 1.03^{20}$	=	3,792 veh/h (LOS E)

While the demand will not surpass the full-hour capacity (SV_E) over the 20 years of the forecast, by year 20, capacity is almost reached. It would be wise, given planning, design and construction time lags, to begin serious planning for a solution in year 10 or earlier.

Problem 28-9

This problem concerns a rural recreational freeway, with the peak period of interest occurring when light to moderate snow is falling (winter). Also, because of its recreational nature, drivers are primarily unfamiliar with the route. Because of these two unique features, the solution will involve both a speed adjustment factor (SAF) and a capacity adjustment factor (CAF).

The free-flow speed of the facility must be estimated using Eqn 28-2:

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22 TRD^{0.84}$$

where: f_{LW} = 0.0 mi/h (Table 28-3, 12 ft lanes),
f_{LC} = 0.0 mi/h (Table 28-4, 6 ft clearances), and
TRD = 2 ramps/mi (given).

Then:

$$FFS = 75.4 - 0.0 - 0.0 - 3.22(2^{0.84}) = 69.6\ mi/h$$

However, because of the special circumstances, a speed adjustment factor (SAF) must be applied to this value to account for two non-standard conditions: light to moderate snow, and non-regular users of the facility.

From Table 28-11, the SAF for light-medium snow with a FFS of 69.6 mi/h is 0.86. While the FFS must technically be interpolated, the value is close enough to 70 mi/h to use this value directly. From Table 28-13, the SAF for "all unfamiliar drivers" is 0.863. Then:

$$SAF = 0.86 * 0.863 = 0.742$$
$$FFS_{adj} = FFS * SAF = 69.6 * 0.742 = 51.6\ mi/h$$

For this problem, we will have to calibrate a speed-flow curve for the segment in the form of Equation 28-2:

$$S = FFS_{adj} \qquad v_p \le BP$$
$$S = FFS_{adj} - \left[\frac{\left(FFS_{adj} - \frac{c_{adj}}{45}\right)(v_p - BP)^a}{(c_{adj} - BP)^a}\right] \qquad v_p > BP$$

Table 28-2 is used to compute key values in this equation. Capacity (c) is estimated as:

$$c = 2{,}200 + (FFS - 50) = 2{,}200 + 10(69.6 - 50) = 2{,}396\ \ pc/h/\ln$$

However, this value too must be adjusted using capacity adjustment factors (CAF) to account for light-medium snow and unfamiliar facility users. Note that Table 28-10 is entered with the unadjusted value of FFS to avoid "double counting" the impact on speed. From Table 28-10, the CAF for light-medium snow is 0.90. From Table 28-13, the CAF for "all unfamiliar users" is 0.852. Then:

$$CAF = 0.90 * 0.852 = 0.7668$$
$$c_{adj} = c * CAF = 2{,}396 * 0.7668 = 1{,}837\ \ pc/h/\ln$$

The breakpoint (BP) in the equation is estimated as:

$$BP = [1000 + 40(75 - FFS_{adj})] * CAF^2$$
$$BP = [1000 + 40(75 - 51.6)] * 0.7668^2 = 1{,}138 \ pc/h/\ln$$

From Table 28-2 directly, a = 2 for freeways. Then:

$$S = 51.6 \qquad v_p \le 1{,}138$$
$$S = 51.6 - \left[\frac{\left(51.6 - \frac{1837}{45}\right)(v_p - 1138)^2}{(1837 - 1138)^2} \right] \qquad v_p > 1{,}138$$
$$S = 51.6 - \left[\frac{10.78(v_p - 1138)^2}{481{,}601} \right] \qquad v_p > 1{,}138$$

To find the expected speed of the traffic stream for the prevailing conditions described, the demand volume of 4,000 veh/h must be adjusted to reflect a flow rate per lane in pc/h/ln. This is accomplished using Equation 28-11:

$$v_p = \frac{V}{PHF * N * f_{HV}}$$

The value of V is given (4,000 veh/h), as is N (3 lanes in one direction) and PHF (0.95). The value of f_{HV} is based on the value of E_{HV}, obtained from Table 28-17 for level terrain (2.0). Then:

$$f_{HV} = \frac{1}{1 + 0.10(2.0 - 1)} = 0.909$$
$$v_p = \frac{4{,}000}{0.95 * 3 * 0.909} = 1{,}544 \ pc/h/\ln$$

As 1,544 pc/h/ln is greater than 1,138 pc/h/ln (the breakpoint in the equation), the second part of the speed-flow equation is used to determine the expect speed of the traffic stream:

$$S = 51.6 - \left[\frac{10.78(1544 - 1138)^2}{481{,}601} \right] = 47.9 \ mi/h$$

The density of the traffic stream may now be computed as:

$$D = {}^{v_p}\!/_{S} = {}^{1544}\!/_{47.9} = 32.2 \ pc/h/\ln$$

From Table 28-1, this corresponds to LOS D, but is close to the LOS D/E boundary. Operations are expected to be relatively poor, but stable. The base conditions of poor weather combined with an unfamiliar driver population has a tremendous impact on the situation.

Problem 28-10

The capacity of the work zone is estimated using Equations 28-6 and 28-7. The first estimates the queue discharge rate from the work zone when congested, and the second estimates the capacity from the discharge rate:

$$QDR_{WZ} = 2{,}093 - (154\,LCSI) - (194\, f_{BR}) - (179\, f_{AT}) + (9 f_{LAT}) - (59\, f_{DN})$$

where:
- $LCSI$ = 0.75 (Table 28-14, 3 lanes to 2 lanes),
- f_{BR} = 0 (concrete barrier),
- f_{AT} = 1 (rural area),
- f_{LAT} = 0 ft (given), and
- f_{DN} = 0 (daytime).

Then:

$$QDR_{WZ} = 2093 - (154 * 0.75) - (194 * 0) - (179 * 1) + (9 * 0) - (59 * 0) = 1{,}799 \;\; pc/h/\ln$$

and:

$$c_{WZ} = QDR_{WZ}\left(\frac{100}{100-\alpha_{WZ}}\right) = 1799\left(\frac{100}{100-13.4}\right) = 2{,}077 \;\; pc/h/\ln$$

where 13.4% is the default value for α_{WZ}.

Because there are two lanes open in the work zone, the capacity will be 2077*2 = 4,154 pc/h. Note that this is still expressed as a flow rate in pc/h.

The free-flow speed of the work zone is estimated using Equation 28-9:

$$FFS_{WZ} = 9.95 + (33.49\, f_{SR}) + (0.53 SL_{WZ}) - (5.60\, LCSI) - (3.84\, f_{BR}) - (1.71\, f_{DN}) - (8.7\, TRD)$$

where:
- f_{SR} = 70/50 = 1.4,
- SL_{WZ} = 50 mi/h (given),
- $LCSI$ = 0.75 (as above),
- f_{BR} = 0 (as above),
- f_{DN} = 0 (as above), and
- TRD = 1 ramp/mi (given).

Then:

$$FFS_{WZ} = 9.95 + (33.49 * 1.40) + (0.53 * 50) - (5.60 * 0.75) - (3.84 * 0) - (1.71 * 0) - (8.7 * 1)$$
$$FFS_{WZ} = 70.4\ mi/h$$

The predicted FFS of the work zone is high, especially when compared to the relative speed limits of the freeway and the work zone. This suggests that drivers are not slowing down very much in response to the work zone during periods of light flow.

A speed-flow equation for the work zone could be calibrated using the form of Equation 28-1. It would use the capacity (per lane) and the FFS of the work zone as inputs. This is not required for this problem, and is not done in this case.

Solutions to Problems in Chapter 29

Capacity and Level of Service Analysis: Weaving Segments on Freeways and Multilane Highways

Problem 29-1

The weaving segment shown is a classic ramp-weave formed by a one-lane on-ramp followed by a one-lane off-ramp. As shown in Figure 29-4 (a), for such a ramp-weave, the following values quantify the configuration:

- $LC_{RF} = 1$
- $LC_{FR} = 1$
- $N_{WL} = 2$

Step 1 – Convert Demand Volumes to Flow Rates in pc/h

Each of the volumes in the segment diagram must be converted to a flow rate in pc/h, based upon a PHF = 0.95, rolling terrain, and the percent trucks shown in the problem statement. It is assumed that the standard mix of trucks applies. The conversion is accomplished using Equation 29-1:

$$v = \frac{V}{PHF * f_{HV}}$$

where: E_{HV} = 2 (Table 28-1, Ch 28)
P_{HV} = 0.07 (freeway), 0.03 (on-ramp), 0.05 (off-ramp)

Then:

$$f_{HV} = \frac{1}{1 + P_{HV}(E_{HV} - 1)}$$

$$f_{HV,\,freeway} = \frac{1}{1 + 0.07(2-1)} = 0.935$$

$$f_{HV,on-ramp} = \frac{1}{1 + 0.03(2-1)} = 0.971$$

$$f_{HV,off-ramp} = \frac{1}{1 + 0.05(2-1)} = 0.952$$

and:

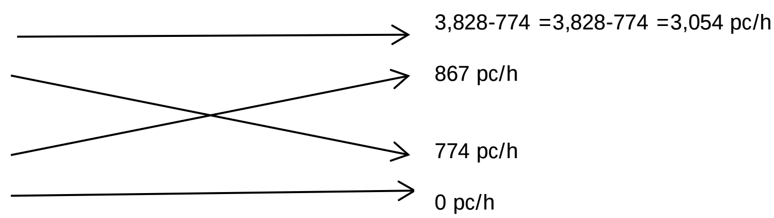
$$v_{freeway} = \frac{3400}{0.95 * 0.935} = 3{,}828 \ pc/h$$

$$v_{on-ramp} = \frac{800}{0.95 * 0.971} = 867 \ pc/h$$

$$v_{off-ramp} = \frac{700}{0.95 * 0.952} = 774 \ pc/h$$

Step 2: Construct the Weaving Diagram

In this problem, the demand flows are not given in the form of a weaving diagram. In this format, it is important to note that the off-ramp demand volume is *included* in the entering freeway volume. Then:



Step 3: Compute Key Configuration Variables

The following values, some of which quantify the effects of configuration, are used throughout the solution:

- v_W = 867 + 774 = 1,641pc/h
- v_{NW} = 3,054 pc/h
- v = 1,641 + 3,054 = 4,695 pc/h
- VR = 1641/4695 = 0.350
- L_S = 1,600 ft

The last configuration characteristic is LC_{MIN}, the minimum number of lane changes needed for weaving vehicles to successfully complete their maneuvers. It is estimated using Equation 29-2:

$$LC_{MIN} = (LC_{FR} * v_{FR}) + (LC_{RF} * v_{RF}) = (1*774) + (1*867) = 1{,}641\ lc/h$$

Step 4: Determine the Maximum Allowable Weaving Length

The maximum weaving length is given by Equation 29-4:

$$L_{MAX} = \left[5{,}728(1+VR)^{1.6}\right] - \left[1{,}566\,N_{WV}\right]$$
$$L_{MAX} = \left[5{,}728*1.35^{1.6}\right] - \left[1{,}566*2\right] = 12{,}390\ ft$$

As this is longer than the existing 1,600-ft length of the segment, it can be analyzed as a weaving segment, and the computations will continue.

Step 5: Determine the Capacity of the Weaving Segment

The capacity of the weaving area can be estimated in two ways. The first is based on the assumption that weaving areas will break down at a density of approximately 43 pc/h/ln. Equation 29-5 is used:

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765\,L_S\right] + \left[119.8\,N_{WV}\right]$$

The value of c_{IFL} is taken from Table 29-2 for a FFS of 65 mi/h (given). For a 65-mi/h freeway, c_{IFL} is 2,350 pc/h/ln. Other values are as previously determined or given:

$$c_{IWL} = 2{,}350 - \left[438.2(1+0.35)^{1.6}\right] + \left[0.0765*1{,}600\right] + \left[119.8*2\right]$$
$$c_{IWL} = 2{,}350 - 708.8 + 122.4 + 239.6 = 2{,}003 \ pc/h/\ln$$

Given that there are 4 lanes in the weaving segment, its capacity is 4*2003 = 8,012 pc/h.

The second way to estimate capacity is based upon the limits of weaving segments to accommodate weaving traffic. This is done using Equation 29-7, with N_{WL} = 2:

$$c_{IW} = \frac{2{,}400}{VR} = \frac{2{,}400}{0.35} = 6{,}857 \ pc/h$$

The smaller of the estimates holds, in this case 6,857 pc/h. As the controlling value is weaving traffic, it is possible that when the weaving segment reaches capacity, there will be unused capacity in the outer lanes of the freeway.

We must now check to insure that capacity is larger than the total demand. We can convert the capacity to veh/h as a full-hour volume, and compare it to demand as stated, in full-hour volume. In this case, we have already converted all the volume elements to flow rates in pc/h, so they can be compared directly to the capacity stated on the same basis:

$$v/c = 4{,}695/6{,}857 = 0.685$$

Obviously, there is no capacity deficiency present, and the solution may continue to determine the level of service.

<u>Step 6: Determine Lane-Changing Rates for Weaving and Non-Weaving Vehicles</u>
The minimum number of lane changes that weaving vehicles will have to make has already been determined as LC_{MIN}. Weaving vehicles, may, however, make additional optional lane changes. The total rate is estimated using Equation 29-12:

$$LC_W = LC_{MIN} + 0.39\left[(L_S - 300)^{0.5} N^2 (1+ID)^{0.8}\right]$$
$$LC_W = 1{,}641 + 0.39\left[(1{,}600 - 300)^{0.5} 4^2 (1+1.6)^{0.8}\right]$$
$$LC_W = 1{,}641 + 0.39*1{,}239 = 2{,}124 \ lc/h$$

The number of lane changes made by non-weaving vehicles is estimated using Equations 29-13. The choice of which equation to use is based upon the lane-changing index, I_{NW}, which is computed using Equation 29-14:

$$I_{NW} = \frac{L_S \, ID v_{NW}}{10{,}000} = \frac{1{,}600 * 1.6 * 3{,}054}{10{,}000} = \frac{1600 * 1.6 * 3054}{10{,}000} = 781.8$$

As this is less than 1,300, the first equation is used:

$$LC_{NW} = (0.206 v_{NW}) + (0.542 L_S) - (192 N)$$
$$LC_{NW} = (0.206 * 3{,}054) + (0.542 * 1{,}600) - (192 * 4)$$
$$LC_{NW} = 629.2 + 867.2 - 768 = 728.4, \ say \ 728 \ lc/h$$

The total lane-changing rate is the sum of the weaving and non-weaving vehicle lane changes:

$$LC_{TOT} = 2{,}124 + 728 = 2{,}852 \ lc/h$$

Step 7: Determine the Speed of Weaving and Non-Weaving Vehicles
The average speed of weaving vehicles in the segment is estimated using Equations 29-18 and 29-19:

$$W = 0.226 \left(\frac{LC_{ALL}}{L_S} \right)^{0.789} = 0.226 \left(\frac{2852}{1600} \right)^{0.789} = 0.357$$
$$S_W = 15 + \left(\frac{FFS * SAF - 15}{1 + W} \right) = 15 + \left(\frac{65 * 1 - 15}{1.357} \right) = 51.8 \ mi/h$$

The average speed of non-weaving vehicles in the segment is estimated using Equation 29-20:

$$S_{NW} = FFS * SAF - (0.0072 LC_{MIN}) - \left(0.0046 * {}^{v}\!/_{N}\right)$$
$$S_{NW} = 65 * 1 - (0.0072 * 1641) - \left(0.0046 * {}^{4695}\!/_{4}\right)$$
$$S_{NW} = 65.0 - 11.8 - 5.5 = 47.7 \ mi/h$$

The average speed of all vehicles is computed using Equation 29-21:

$$S = \frac{v_W + v_{NW}}{\left(\frac{v_W}{S_W} \right) + \left(\frac{v_{NW}}{S_{NW}} \right)} = \frac{1641 + 3054}{\left(\frac{1641}{51.8} \right) + \left(\frac{3054}{47.7} \right)} = \frac{4695}{31.68 + 64.02} = 49.1 \ mi/h$$

Step 8: Determine the Density and Level of Service for the Segment
Density is estimated using Equation 29-22:

$$D = \frac{v/N}{S} = \frac{4695/4}{49.1} = 23.9 \ pc/mi/\ln$$

From Table 29-1, this represents LOS C.

Discussion

This segment operates at a desirable LOS. Densities are low, despite relatively low speeds through the segment, which are considerably under the FFS of 65 mi/h. Thus, drivers will experience the effects of congestion through reduced speeds. Nevertheless, the operation is quite stable, and demands are well below capacity.

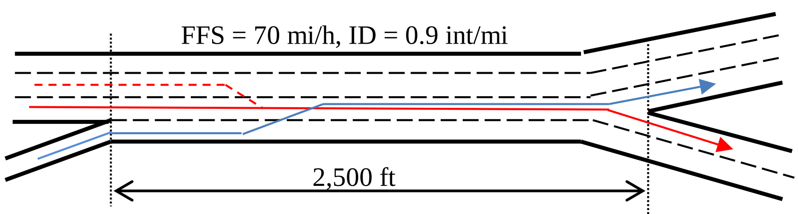
It should be noted that weaving vehicles are traveling faster than non-weaving vehicles. This may reflect non-weaving vehicles crowding left lanes to avoid conflicts with weaving vehicles.

Problem 29-2

(a) ***Basic Characteristics of the Weaving Segment***

The basic characteristics of the weaving segment include geometric and lane-changing characteristics that quantify the segment and its operation. It also includes basic characteristics of the demand, most of which are given in the problem statement.

For key geometric and lane-changing characteristics, Figure 29-12 must be examined in detail, as illustrated below:



For clarity, the right legs of both the entry and exit are designated as the “ramps” for this analysis. The ramp-to-freeway flow (blue), shows that vehicles in this flow must make at least one lane change. The freeway-to-ramp movement can be made without any lane changes. Also, the freeway-to-ramp maneuver may be made with one lane change from the middle freeway lane, and with no lane changes from the right-most freeway lane. Therefore:

$LC_{RF} = 1$
$LC_{FR} = 0$
$N_{WL} = 3$

The following geometric variables are specified in the problem statement:

FFS = 70 mi/h
L_S = 2,500 ft
ID = 0.9 int/mi

Note that there are no barrier lines in the weaving segment, so the short length is equal to the base length in this case.

From the weaving diagram of Figure 29-12, the following demand flows are known:

v_{o1} = 2,000 pc/h
v_{o2} = 1,000 pc/h
v_{NW} = 2,000 + 1,000 = 3,000 pc/h
v_{w1} = 1,600 pc/h
v_{w2} = 800 pc/h
v_W = 1,600 + 800 = 2,400 pc/h
v = 3,000 + 2,400 = 5,400 pc/h
VR = 2,400/5,400 = 0.444

(b) (c) ***Level of Service and Capacity of the Weaving Segment***
Parts b and c of the problem must be addressed simultaneously, as determining the LOS requires a prior determination of capacity.

Equation 29-2 is used to find the minimum number of lane-changes made by weaving vehicles traversing the segment:

$$LC_{MIN} = (LC_{RF} * v_{RF}) + (LC_{FR} * v_{FR})$$
$$LC_{MIN} = (800 * 1) + (1{,}600 * 0) = 800\ lc/h$$

Note that in this case, v_{RF} is v_{w2} and v_{RF} is v_{w1}.

Next, the maximum weaving length is computed to determine whether the segment is indeed a weaving segment. Equation 29-4 is used:

$$L_{MAX} = \left[5{,}728(1+VR)^{1.6}\right] - \left[1{,}566\, N_{WL}\right]$$
$$L_{MAX} = \left[5{,}728(1+0.444)^{1.6}\right] - \left[1{,}566 * 3\right]$$
$$L_{MAX} = 10{,}311 - 4{,}698 = 5{,}614\ ft$$

As this is greater than the existing length of 2,500 ft, the analysis continues as a weaving segment.

The capacity of the weaving segment is now computed using Equations 26-5 through 29-8. Two possible results are obtained. The capacity when controlled by a maximum density of 43 pc/mi/ln is given by Equations 29-5 and 29-6.

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765\,L_S\right] + \left[119.8\,N_{WL}\right]$$

The value of c_{IFL} is obtained from Table 29-2 as 2,400 pc/h/ln. The weaving segment capacity is computed as:

$$c_{IWL} = 2400 - \left[438.2(1.444)^{1.6}\right] + \left[0.0765 * 2500\right] + \left[119.8 * 3\right]$$
$$c_{IWL} = 2400 - 788.8 + 191.3 + 359.4 = 2{,}162\ pc/h/\ln$$
$$c_{IW} = 3 * 2162 = 6{,}486\, pc/h$$

If the capacity for weaving movements controls the situation, the capacity is estimated using Equation 29-7 (for N_{WL} of 3):

$$c_{IW} = \frac{3{,}500}{VR} = \frac{3500}{0.444} = 7{,}883\ pc/h$$

The smaller value, 6,486 pc/h prevails. This is the answer to **part (c)**, the capacity under ideal conditions. It is also larger than the total demand flow rate of 5,400 pc/h. LOS F *does not exist*, and the analysis may continue.

The total lane-changing rate for weaving vehicles is given by Equation 29-12:

$$LC_W = LC_{MIN} + 0.39\left[(L_S - 300)^{0.5}\, N^2\,(1+ID)^{0.8}\right]$$
$$LC_W = 800 + 0.39\left[(2500 - 300)^{0.5}\ 4^2\,(1+0.9)^{0.8}\right]$$
$$LC_W = 800 + 0.39\left[46.9 * 16 * 1.67\right] = 1{,}289\ lc/h$$

The total lane-changing rate for non-weaving vehicles is determined by Equations 29-13. There are two equations. To select the appropriate equation, the value of the lane-changing index must be determined using Equation 29-14:

$$I_{NW} = \frac{L_S\, ID\, v_{NW}}{10{,}000} = \frac{2500 * 0.8 * 3000}{10{,}000} = 675 < 1{,}300$$

Because the index is less than 1,300, the first Equation 29-12 is applicable:

$$LC_{NW} = (0.206\, v_{NW}) + (0.542\, L_S) - (192\, N)$$
$$LC_{NW} = (0.206 * 3000) + (0.542 * 2500) - (192 * 4)$$
$$LC_{NW} = 618 + 1{,}355 - 768 = 1{,}205\ lc/h$$

The total lane-changing in the segment is, therefore:

$$LC_{ALL} = LC_W + LC_{NW} = 1289 + 1205 = 2{,}494\ lc/h$$

The average speed of weaving vehicles in the segment is estimated using Equations 29-18 and 29-19:

$$W = 0.226\left(\frac{LC_{ALL}}{L_S}\right)^{0.789} = 0.226\left(\frac{2494}{2500}\right)^{0.789} = 0.2256$$
$$S_W = 15+\left(\frac{FFS*SAF-15}{1+W}\right) = 15+\left(\frac{70*1-15}{1+0.2256}\right) = 59.9\ mi/h$$

The average speed of non-weaving vehicles is estimated using Equation 29-20:

$$S_{NW} = FFS*SAF-(0.0072\,LC_{MIN})-\left(0.0048\,{}^{v}\!/_{N}\right)$$
$$S_{NW} = 70*1-(0.0072*800)-\left(0.0048*{}^{5400}\!/_{4}\right) = 57.8\ mi/h$$

The average speed of all vehicles is then computed using Equation 29-21:

$$S = \frac{v_W+v_{NW}}{\left(\frac{v_W}{S_w}\right)+\left(\frac{v_{NW}}{S_{NW}}\right)} = \frac{2400+3000}{\left(\frac{2400}{59.9}\right)+\left(\frac{3000}{57.8}\right)} = 58.7\ mi/h$$

The density is then computed as:

$$D = \frac{{}^{v}\!/_{N}}{S} = \frac{{}^{5400}\!/_{4}}{58.7} = 23.0\ pc/mi/\ln$$

From Table 29-1, this is LOS C. Operations are expected to be stable and good.

(d) Capacity Under Prevailing Conditions

The capacity of the segment was previously estimated as 6,486 pc/h. This value, however, assumes "ideal" or base conditions, which include no trucks. To get the capacity in mixed vehicles per hour, this value must be multiplied by the appropriate heavy vehicle adjustment factor, f_{HV}.

The demand includes 10% trucks (standard mix assumed) in rolling terrain. From Table 28-10, for rolling terrain, E_{HV} = 3, and:

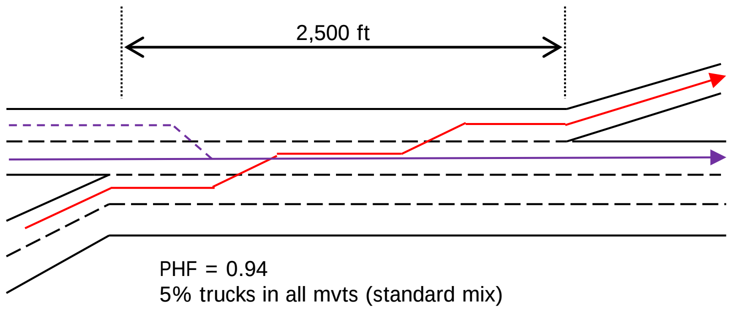
$$f_{HV} = \frac{1}{1+P_{HV}(E_{HV}-1)} = \frac{1}{1+0.10(3-1)} = 0.833$$

Capacity under prevailing conditions is, therefore, 6,486*0.833, or 5,403 vehicles per hour.

Note that the PHF is *not* applied. Capacity, even under prevailing conditions, is always stated as a flow rate. If the maximum full-hour volume that the segment could accommodate were desired, the capacity would be multiplied by the PHF, or 5,403*0.92 = 4,971 veh/h.

Problem 29-3

The configuration of this weaving segment is an interesting one. The first critical question is to classify it as a one-sided or two-sided weaving segment. Then, the critical configuration characteristics would have to be quantified. Close perusal of the segment is required:



At first glance, this might appear to be a two-sided weaving segment, with a right-side on-ramp followed by a left-side off-ramp. However, a closer look reveals that the segment does not meet *either* of the two criteria for two-sided weaving: a one-lane on-ramp followed by a one-lane off-ramp on opposite sides of the roadway (one ramp has two lanes), or one weaving movement requiring *more* the two lane changes. The right-to-left weaving movement does require two lane changes, but does not require *more* than two lane changes. Thus, this is still classified as a one-sided weaving segment.

The standard terminology of one-sided weaves, however, doesn't quite fit either, as the *ramps* are clearly on opposite sides of the freeway. The terminology fits a bit better if the left entry leg is considered as the ramp; then, both "ramps" are on the left side of the freeway. To avoid confusion with notation, that is what we will do for the remainder of the problem.

From the figure, the following key variables can be noted:

LC_{RF} (left entry, right exit) = 0
LC_{FR} (right entry, left exit) = 2
N_{WL} = 3

Note that as there are no barrier markings in the segment, the base length and the short length are the same, or 2,500 ft.

Demand volumes must all be converted to equivalent flow rates in pc/h. The PHF is given (0.94) and there are 5% trucks (standard mix) in level terrain. From Table 28-10, for level terrain, E_{HV} = 2, and:

$$f_{HV} = \frac{1}{1+P_{HV}\left(E_{HV}-1\right)} = \frac{1}{1+0.05\left(2-1\right)} = 0.952$$

Then, using Equation 29-1:

$$v = \frac{V}{PHF * f_{HV}}$$
$$v_{o1(FF)} = \frac{2000}{0.94*0.952} = 2{,}235 \ pc/h$$
$$v_{o2(RR)} = \frac{800}{0.94*0.952} = 894 \ pc/h$$
$$v_{w1(RF)} = \frac{1300}{0.94*0.952} = 1{,}493 \ pc/h$$
$$v_{w2(FR)} = \frac{500}{0.94*0.952} = 559 \ veh/h$$

From these values, the following additional key variables may be determined:

v_W = 1493 + 559 = 2,052 pc/h
v_{NW} = 2235 + 894 = 3,129 pc/h
v = 2052 + 3129 = 5,181 pc/h
VR = 2052/5181 = 0.396

Equation 29-2 is used to find the minimum number of lane changes needed for weaving vehicles to complete their weaving maneuvers:

$$LC_{MIN} = \left(LC_{RF} * v_{RF}\right) + \left(LC_{FR} * v_{FR}\right) = \left(0*1493\right) + \left(2*559\right) = 1{,}118 \ lc/h$$

The maximum weaving length is given by Equation 29-4:

$$L_{MAX} = \left[5{,}728\left(1+VR\right)^{1.6}\right] - \left[1{,}566\,N_{WV}\right]$$
$$L_{MAX} = \left[5{,}728\left(1+0.396\right)^{1.6}\right] - \left[1{,}566*3\right] = 9768.3 - 4698.0 = 5{,}070.3 \ ft$$

As this is longer than the actual length of 2,500 ft, the segment is operating as a weaving segment, and the analysis on this basis may continue.

The next key determination is the capacity of the weaving segment. The first possible determinant of capacity is based upon the breakdown density of 43 pc/mi/ln being reached. This capacity is estimated using Equations 29-5 and 29-6:

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765(L_S)\right] + \left[119.8\,N_{WV}\right]$$

The value of c_{IFL} is obtained from Table 29-2 for a FFS of 60 mi/h: 2,300 pc/h/ln. Then:

$$\begin{aligned}
c_{IWL} &= 2{,}300 - \left[438.2 * 1.396^{1.6}\right] + \left[0.0765 * 2500\right] + \left[119.8 * 3\right] \\
c_{IWL} &= 2{,}300 - 747.3 + 191.3 + 357.9 = 2{,}102\ pc/h/\ln \\
c_W &= c_{IWL} * N * f_{HV} = 2102 * 4 * 0.952 = 8{,}004\ veh/h
\end{aligned}$$

The second potential determinant of capacity is the limiting weaving flow rates, as specified in Equations 29-7. The equation for N_{WV} = 3 is used:

$$\begin{aligned}
c_{IW} &= \frac{3500}{VR} = \frac{3500}{0.396} = 8{,}838\ pc/h \\
c_W &= c_{IW} * f_{HV} = 8838 * 0.952 = 8{,}414\ veh/h
\end{aligned}$$

The smaller value, or 8,004 veh/h is the capacity of the weaving segment. This must be compared to the total demand flow rate. The total demand volume in veh/h is 4,600 veh/h. To compare it to capacity directly, this has to be converted to a flow rate using the PHF, or 4600/PHF = 4600/0.94 = 4,894 veh/h. This is significantly lower than the capacity, so LOS F does not exist. The analysis may continue.

The total lane-changing rate for weaving vehicles is estimated using Equation 29-12:

$$\begin{aligned}
LC_W &= LC_{MIN} + 0.39\left[(L_S - 300)^{0.5}\, N^2 (1 + ID)^{0.8}\right] \\
LC_W &= 1118 + 0.39\left[(2500 - 300)^{0.5}\, 4^2 (1+2)^{0.8}\right] = 1{,}823\ lc/h
\end{aligned}$$

The total lane-changing rate for non-weaving vehicles is estimated using Equations 29-13, based upon the lane-changing index computed using Equation 29-14:

$$I_{NW} = \frac{L_S\, ID\, v_{NW}}{10{,}000} = \frac{2500 * 2 * 3129}{10{,}000} = 1{,}564.5$$

Because this index is in the middle range of 1300 to 1950, the actual result is interpolated between the two Equations 29-13, as indicated in Equation 29-15. Then:

$$LC_{NW1} = (0.206\,v_{NW}) + (0.542\,L_S) - (192.6\,N)$$
$$LC_{NW1} = (0.206*3129) + (0.542*2500) - (192.6*4) = 1{,}229\ lc/h$$
$$LC_{NW2} = 2{,}135 + 0.223(v_{NW} - 2{,}000) = 2135 + 0.223(3129 - 2000) = 2{,}387\ lc/h$$
$$LC_{NW} = LC_{W1} + (LC_{W2} - LC_{W1})\left(\frac{I_{NW} - 1300}{650}\right) = 1229 + (2387 - 1229)\left(\frac{1564.5 - 1300}{650}\right)$$
$$LC_{NW} = 1229 + 1158\left(\frac{264.5}{650}\right) = 1{,}700\ lc/h$$

The total lane-changing rate for the weaving segment is, therefore:

$$LC_{ALL} = LC_W + LC_{NW} = 1823 + 1700 = 3{,}523\ lc/h$$

The average speed of weaving vehicles in the weaving segment is given by Equations 29-18 and 29-19:

$$W = 0.226\left(\frac{LC_{ALL}}{L_S}\right)^{0.789} = 0.226\left(\frac{3523}{2500}\right)^{0.789} = 0.2923$$
$$S_W = 15 + \left(\frac{FFS*SAF - 15}{1 + W}\right) = 15 + \left(\frac{60*1 - 15}{1 + 0.2923}\right) = 49.8\ mi/h$$

The average speed of non-weaving vehicles is given by Equation 29-20:

$$S_{NW} = FFS*SAF - (0.0072\,LC_{MIN}) - \left(0.0048\, v/N\right)$$
$$S_{NW} = 60*1 - (0.0072*1118) - \left(0.0048*5181/4\right) = 45.7\ mi/h$$

The average speed for all vehicles may now be computed using Equation 29-21:

$$S = \frac{v_W + v_{NW}}{\left(\frac{v_W}{S_W}\right) + \left(\frac{v_{NW}}{S_{NW}}\right)} = \frac{2052 + 3129}{\left(\frac{2052}{49.8}\right) + \left(\frac{3129}{45.7}\right)} = 47.2\ mi/h$$

The density may now be computed as:

$$D = \frac{v/N}{S} = \frac{5181/4}{47.2} = 27.4\ pc/mi/\ln$$

From Table 29-1, this is LOS C, but very close to the LOS C/D boundary of 28 pc/mi/ln.

Weaving vehicles travel faster than non-weaving vehicles in this segment. For the geometry and flows shown, this is logical, as one of the “weaving” flows looks more like a through freeway flow, much as in a two-sided weaving segment (even though this is still a one-sided weaving segment). A LOS C determination is usually acceptable, but the closeness of this operation to the LOS D boundary means that growth potential must be carefully monitored over the years.

Problem 29-4

The segment shown in Figure 29-14 is a ramp-weave located on a C-D (collector-distributor) roadway as part of a freeway interchange. It can be analyzed as a weaving segment, but several key determinations will have to be considered as broad approximations, as the algorithms were all calibrated for mainline freeway weaving segments. For the purposes of terminology, the "ramps" are the left entry and exit points, and the "freeway" consists of the right entry and exit points. Because this is a ramp-weave, key configuration characteristics include:

LC_{RF} = 1
LC_{FR} = 1
N_{WV} = 2

As there are no barrier markings shown, the basic length and the short length of the weaving segment are equal, i.e. 1,200 ft.

Each of the component demand volumes must be converted to a flow rate in pc/h. The PHF is given as 0.92. There are 10% trucks (standard mix) in level terrain. From Table 28-10, E_{HV} = 2, and:

$$f_{HV} = \frac{1}{1+P_{HV}(E_{HV}-1)} = \frac{1}{1+0.10(2-1)} = 0.909$$

Then, using Equation 29-1:

$$v = \frac{V}{PHF * f_{HV}}$$
$$v_{o1(FF)} = \frac{400}{0.92*0.909} = 483\ pc/h$$
$$v_{o2(RR)} = \frac{300}{0.91*0.909} = 363\ pc/h$$
$$v_{w1(FR)} = \frac{800}{0.91*0.909} = 967\ pc/h$$
$$v_{w2(RF)} = \frac{600}{0.91*0.909} = 725\ pc/h$$

Other key variables may now be computed:

$$v_W = 967 + 725 = 1{,}692 \ pc/h$$
$$v_{NW} = 483 + 363 = 846 \ pc/h$$
$$v = 1692 + 846 = 2{,}538 \ pc/h$$
$$VR = 1692/2538 = 0.667$$

The minimum number of lane changes that must be made by weaving vehicles to successfully complete their desired maneuvers is given by Equation 29-2:

$$LC_{MIN} = (LC_{FR} * v_{FR}) + (LC_{RF} * v_{RF}) = (1*967) + (1*725) = 1{,}692 \ lc/h$$

The maximum length of for a weaving segment with this configuration and demand is given by Equation 29-4:

$$L_{MAX} = \left[5{,}728(1+VR)^{1.6}\right] - \left[1{,}566\, N_{WV}\right]$$
$$L_{MAX} = \left[5{,}728(1+0.667)^{1.6}\right] - \left[1{,}566 * 2\right]$$
$$L_{MAX} = 12{,}975 - 3{,}132 = 9{,}843 \ ft > 1{,}200 \ ft$$

Because the actual length is less than the maximum, this segment is operating as a weaving segment, and the analysis continues.

The capacity of the segment can be estimated in two ways. The first is based upon reaching the breakdown density of 43 pc/mi/ln, and is estimated using Equation 29-5:

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765\, L_S\right] + \left[119.8\, N_{WV}\right]$$

The value of c_{IFL} is taken from Table 29-2 for a C-D roadway with a FFS of 55 mi/h. The appropriate value is 2,100 pc/h/ln. Then:

$$c_{IWL} = 2100 - \left[438.2 * 1.667^{1.6}\right] + \left[0.0765 * 1200\right] + \left[119.8 * 2\right]$$
$$c_{IWL} = 2100 - 992.6 + 91.8 + 239.6 = 1{,}439 \ pc/h/\ln$$

With 2 lanes, the segment capacity (under ideal conditions) is 2*1449 = 2,878 pc/h.

The second estimate of capacity is based upon the maximum allowable weaving flow rate in the segment, given by Equations 29-7. Choosing the equation for N_{WV} = 2, the estimate becomes:

$$c_{IW} = \frac{2{,}400}{VR} = \frac{2{,}400}{0.667} = 3{,}598 \ pc/h$$

Obviously, the minimum value, or 2,878 pc/h controls the result. As this is greater than the demand flow rate of 2,538 pc/h, no breakdown is expected and the analysis continues.

The total lane-changing rate for weaving vehicles is estimated using Equation 29-12:

$$LC_W = LC_{MIN} + 0.39\left[(L_S - 300)^{0.5} N^2 (1+ID)^{0.8}\right]$$
$$LC_W = 1{,}692 + 0.39\left[(1{,}200 - 300)^{0.5} * 2^2 * 1.667^{0.8}\right] = 1{,}762\, lc/h$$

Note that the result is only slightly more than LC_{MIN}, which is logical given the configuration. There is not much opportunity for weaving vehicles to make optional lane changes.

Total lane-changing by non-weaving vehicles is estimated using Equations 29-13. The choice of which equation to use is based upon the lane-changing index, computed using Equation 29-14:

$$I_{NW} = \frac{L_S\, ID\, v_{NW}}{10{,}000} = \frac{1200 * 4 * 846}{10{,}000} = 406 < 1{,}300$$

As the index is less than 1,300, the first Equation 29-13 is used:

$$LC_{NW} = (0.206\, v_{NW}) + (0.542\, L_S) - (192.6\, N)$$
$$LC_{NW} = (0.206 * 846) + (0.542 * 1{,}200) - (192.6 * 2) = 439\ lc/h$$

The result for non-weaving vehicle lane changes is low. Even the low value predicted, however, might be too high. From the configuration diagram (Figure 29-14), non-weaving vehicles would have little opportunity and little reason to make a lane change at all. The equation, however, was calibrated for freeway mainlines, and may be a bit inaccurate for the C-D case under study.

The total lane-changing rate, LC_{ALL} = 1762+439=2,201 lc/h.

The average speed of weaving vehicles traversing the segment is estimated using Equations 29-18 and 29-19:

$$W = 0.226\left(\frac{LC_{ALL}}{L_S}\right)^{0.789} = 0.226\left(\frac{2201}{1200}\right)^{0.789} = 0.347$$
$$S_W = 15 + \left(\frac{FFS * SAF - 15}{1+W}\right) = 15 + \left(\frac{55 * 1 - 15}{1 + 0.347}\right) = 44.7\ mi/h$$

The average speed of non-weaving vehicles traversing the weaving segment is estimated using Equation 29-20:

$$S_{NW} = FFS * SAF - (0.0072\, LC_{MIN}) - \left(0.0048\, {}^{v}\!/_{N}\right)$$
$$S_{NW} = 55*1 - (0.0072*1692) - \left(0.0048 * {}^{2538}\!/_{2}\right)$$
$$S_{NW} = 55.0 - 12.2 - 6.1 = 36.7\ mi/h$$

Note that non-weaving vehicles travel at significantly lower speeds than weaving vehicles. The dominant flows weave, and non-weaving flows have no option to avoid the turbulence caused by weaving vehicles. Non-weaving vehicles must share lanes with weaving vehicles, and are therefore affected by them.

The average speed of all vehicles is given by Equation 29-21:

$$S = \frac{v_W + v_{NW}}{\left(\frac{v_W}{S_W}\right) + \left(\frac{v_{NW}}{S_{NW}}\right)} = \frac{1692 + 846}{\left(\frac{1692}{44.7}\right) + \left(\frac{846}{36.7}\right)} = \frac{2538}{37.85 + 23.05} = 39.7\ mi/h$$

The density may now be computed as:

$$D = \frac{{}^{v}\!/_{N}}{S} = \frac{{}^{2538}\!/_{2}}{39.7} = 32.0\ pc/h/\ln$$

From Table 29-1, for a C-D roadway operating at a density of 32 pc/h/ln, the LOS is C – but barely. The density is right at the maximum value for LOS C.

The LOS is minimally acceptable, given that this is a C-D roadway, not a freeway mainline. The speeds through the segment are relatively slow, as would be expected in a C-D situation with the dominant flows weaving.

Solutions to Problems in Chapter 30

Capacity and Level of Service Analysis: Merging and Diverging Segments on Freeways and Multilane Highways

Problem 30-1

The ramp sequence shown is a one-lane on-ramp followed by a one-lane off-ramp, 2,000 ft apart. Consideration of the interaction between the two will be an important part of this analysis.

Step 1: Convert Demand Volumes to Flow Rates in pc/h

Equation 30-1 is used to convert volumes to flow rates in pc/h:

$$v = \frac{V}{PHF * f_{HV}}$$

The peak hour factor (PHF) for all movements is given as 0.95. Rolling terrain prevails. From Chapter 28, the passenger-car equivalent for heavy vehicles in rolling terrain is 3.0. The heavy-vehicle adjustment factor, f_{HV}, depends upon the proportion of trucks in the demand, which varies for each component:

$$f_{HV} = \frac{1}{1 + P_{HV}(E_{HV} - 1)}$$

$$f_{HV,freeway} = \frac{1}{1 + 0.07(3-1)} = 0.877$$

$$f_{HV,Ramp1} = \frac{1}{1 + 0.03(3-1)} = 0.943$$

$$f_{HV,Ramp2} = \frac{1}{1 + 0.05(3-1)} = 0.909$$

Then:

$$v_{F1} = \frac{3400}{0.95 * 0.877} = 4{,}081 \ pc/h$$

$$v_{R1} = \frac{800}{0.95 * 0.943} = 893 \ pc/h$$

$$v_{R2} = \frac{700}{0.95 * 0.909} = 811 \ pc/h$$

Note that the freeway flow rate immediately upstream of the off-ramp is the sum of the entering freeway flow *plus* the on-ramp flow. Thus:

$$v_{F2} = v_{F1} + v_{R1} = 4{,}081 + 893 = 4{,}974 \ pc/h$$

Step 2: Determine the Flow Rate in Lanes 1 and 2 Immediately Upstream of Each Ramp

The real issue involves determining which equation governs the value of v_{12} for each ramp.

Ramp 1

From Table 30-3, an on-ramp with a downstream off-ramp on a 6-lane freeway is governed by either Equation 30-6 or 30-5 in Table 30-2. The determination of which one is applicable depends upon the equivalence distance, L_{EQ}. From Table 30-4, Equation 30-15 is used to compute the equivalence distance:

$$L_{EQ} = 0.214(v_F + v_R) + 0.444L_a + 53.32\,RFFS - 2{,}400$$
$$L_{EQ} = 0.214(4081 + 893) + (0.444 * 500) + (53.32 * 35) - 2{,}400$$
$$L_{EQ} = 1{,}064.4 + 222.0 + 1{,}866.2 - 2{,}400.0 = 752.6\ ft < 2{,}000\ ft$$

Because the actual distance between the ramps (2,000 ft) is *more* than the equivalence distance (752.6 ft), the on-ramp *is not* influenced by the downstream off-ramp, and the general equation, 30-5, is used to determine $v_{12(1)}$. Then:

$$P_{FM} = 0.5775 + 0.000028L_a = 0.5775 + (0.000028 * 500) = 0.5915$$

Then, using Equation 30-2:

$$v_{12} = v_F\, P_{FM}$$
$$v_{12(1)} = 4{,}081 * 0.5915 = 2{,}414\ pc/h$$

This value must be checked for reasonableness. The flow in Lane 3 (there is only one outer lane on a 6-lane freeway) is 4081-2414 = 1,604 pc/h < 2,700 pc/h. It is also less than 1.5 (2414/2) = 1,811. Therefore, the predicted value is considered reasonable, and will be used.

Ramp 2:

From Table 30-3, for an off-ramp with an upstream on-ramp on a 6-lane freeway, v_{12} may be computed using either Equation 30-12 or 30-11. The choice of which is appropriate is once again made by comparing the actual distance between the ramps to the equivalence distance, L_{EQ}. From Table 30-4, the equivalence distance is estimated using Equation 30-17:

$$L_{EQ} = \frac{v_U}{0.071 + 0.000023v_F - 0.000076v_R}$$
$$L_{EQ} = \frac{893}{0.071 + (0.000023 * 4974) - (0.000076 * 811)}$$
$$L_{EQ} = \frac{893}{0.071 + 0.1144 - 0.0616} = \frac{893}{0.1238} = 7{,}213.2\ ft > 2{,}000\ ft$$

Because the equivalence distance (7,213.2 ft) is larger than the actual distance between the ramps (2,000 ft), the off-ramp *is* affected by the upstream on-ramp, and Equation 30-12 is used. Then:

$$P_{FD} = 0.717 - 0.000039 v_F + 0.604 \left(v_u / L_{up} \right)$$
$$P_{FD} = 0.717 - (0.000039 * 4974) + (0.604 * 893 / 2000)$$
$$P_{FD} = 0.717 - 0.1940 + 0.2697 = 0.7927$$

Then, using Equation 30-3:

$$v_{12} = v_R + (v_F - v_R) P_{FD}$$
$$v_{12(2)} = 811 + (4974 - 811) * 0.7927 = 4{,}111 \ pc/h$$

This value must be checked for reasonableness. The flow in the one outer lane (Lane 3) is 4974-4111 = 863 pc/h. This is less than 2,700 pc/h. It is also less than 1.5 (4111/2) = 3,018 pc/h. Therefore, the value is accepted as reasonable.

Some discussion is necessary here. The "reasonableness" check guards against assigning too much traffic to the outer lane. In this case, it is likely that too few have been assigned, given the far higher flows in lanes 1 and 2. We do not correct for this because the resulting LOS will likely be worse than what actually occurs. Thus, any error is on the "safe" side, and the computations continue.

Step 3: Check Capacity Limitations

Capacity limitations are given in Table 30-5. Absolute capacities are given for the freeway and the ramp roadways. Limitations on traffic entering the ramp influence area (Lanes 1 and 2) are also given, but failure of one of these alone does not lead to LOS F. The critical point for the freeway check is between the ramps (v_{F2}), as freeway traffic is at its maximum at this point. These checks are summarized in the table below.

Capacity Checks for Problem 30-1

Element	Demand Flow Rate (pc/h)	Capacity (pc/h)	OK?
Freeway, v_{F2}	4,974	7,050 (Table 28-4, 3 lanes, FFS = 65)	Yes
Ramp 1, v_{R1}	893	2,000 (Table 28-4, RFFS = 35)	Yes
Ramp 2, v_{R2}	811	2,100 (Table 28-4, RFFS = 50)	Yes
Ramp 1 Influence Area, $v_{R12(1)}$	2414+893 = 3,307	4,600 (Table 28-4)	Yes
Ramp 2 Influence Area, $v_{12(2)}$	4,111	4,400 (Table 28-4)	Yes

All of the capacity checks are passed, so LOS F *will not* prevail, the solution continues to find the Level of Service.

Step 4: Determine the Density in the Ramp Influence Areas and the Resulting LOS

The density in the merge area is computed using Equation 30-21. The density in the diverge area is computed using Equation 30-22. Then:

$$D_{Ramp1} = 5.475 + 0.00734v_R + 0.0078v_{12} - 0.00627L_a$$
$$D_{Ramp1} = 5.475 + (0.00734*893) + (0.0078*2414) - (0.0062*500)$$
$$D_{Ramp1} = 5.475 + 6.555 + 18.829 - 3.100 = 27.76\ pc/mi/\ln$$
$$D_{Ramp2} = 4.252 + 0.0086v_{12} - 0.009L_a$$
$$D_{Ramp2} = 4.252 + (0.0086*4111) - (0.009*300)$$
$$D_{Ramp2} = 4.252 + 35.355 - 2.700 = 38.21\ pc/mi/\ln$$

From Table 30-1, Ramp 1 is operating at LOS C (just barely, with a maximum of 28.0 pc/mi/ln), while Ramp 2 is operating at LOS E.

Step 5: Speed Characteristics
Equations for the prediction of speed are given in Table 30-6 for merge segments, and in Table 30-7 for diverge segments.

For Ramp 1 (merge segment):

$$M_S = 0.321 + 0.0039e^{(v_{R12}/1000)} - 0.002(L_a * RFFS/1000)$$
$$M_S = 0.321 + (0.0039 * e^{3307/1000}) - (0.002*500*35/1000)$$
$$M_S = 0.321 + 0.1065 - 0.035 = 0.3925$$
$$S_{R1} = FFS*SAF - (FFS*SAF - 42)M_S$$
$$S_{R1} = 65*1 - (65*1 - 42)*0.3925 = 65.0 - 9.0275 = 56.0\ mi/h$$
$$S_o = FFS*SAF - 0.0036(v_{oa} - 500) = 65*1 - 0.0036(1667 - 500) = 60.8\ mi/h$$
$$S = \frac{v_{R12} + v_{oa}N_o}{\left(\frac{v_{R12}}{S_R}\right) + \left(\frac{v_{oa}N_o}{S_o}\right)} = \frac{3307 + 1667*1}{\left(\frac{3307}{56.0}\right) + \left(\frac{1667*1}{60.8}\right)} = \frac{4974}{59.05 + 27.42} = 57.5\ mi/h$$

For Ramp 2 (diverge segment)

$$D_S = 0.883 + 0.00009v_{12} - 0.013RFFS$$
$$D_S = 0.833 + (0.00009*4111) - (0.013*50)$$
$$D_S = 0.833 + 0.3670 - 0.6500 = 0.550$$
$$S_R = 1.097FFS*SAF - (FFS*SAF - 42)D_S$$
$$S_R = (1.097*65*1) - (65*1 - 42)*0.550$$
$$S_R = 71.31 - 12.65 = 58.7\ mi/h$$
$$S_o = 1.097FFS*SAF = 1.097*65*1 = 71.3\ mi/h$$
$$S = \frac{v_{12} + v_{oa}N_0}{\left(\frac{v_{12}}{S_R}\right) + \left(\frac{v_{oa}N_o}{S_o}\right)} = \frac{4111 + 863*1}{\left(\frac{4111}{58.7}\right) + \left(\frac{863*1}{71.3}\right)} = \frac{4974}{70.03 + 12.10} = 60.4\,mi/h$$

Analysis
For the ramp sequence as shown, the off-ramp is clearly the critical part of the operation. Its density places it at LOS E. However, the speed behavior at the off-ramp seems to be a bit better than that at the on-ramp. There are two possible reasons for this. Speeds at off-ramps are generally higher than for on-ramps at similar demand flow rates. The outer speed is also very high, based upon a very small allocation of demand to the outer flow. It is likely that the actual distribution of vehicles will put more vehicles in the outer lane,

and reduce the predicted speed. Note that the influence areas of the ramps overlap, so LOC E prevails for 1,500 ft upstream of the off-ramp.

Part (b) of the question asks to compare this to the solution to Problem 29-1, which was fundamentally the same segment with a continuous auxiliary lane, creating a ramp-weave. For that solution, the LOS was C (better than the ramps in this solution), but the average speed was quite a bit lower (49.1 mi/h). The weaving segment essentially spreads the turbulence throughout the 2,000 ft distance between ramps, whereas the ramp configuration focuses more of it closer to the ramps. Given the overlap of the ramp influence area in this case, that would not be expected to be a major factor, however.

In the final analysis, the methodologies take very different approaches, and direct comparison is a bit difficult. Distribution of demand flows over the lanes of a weaving segment is not a critical factor in weaving analysis, whereas it dominates ramp analysis. It would be a difficult call to make, but one suspects that the continuous auxiliary lane would improve operations somewhat.

Problem 30-2

Problem 30-2 is a simple isolated on-ramp on a 6-lane freeway. The analysis is relatively straightforward, but *two* analyses will be necessary, as the question is asking for the impact of an increase in the on-ramp traffic due to a new development. For simplicity, the two analyses will be done simultaneously, so that intermediate computations may be compared as well as the resulting LOS.

Step 1: Convert Demand Volumes to Flow Rates in pc/h

Equation 30-1 is used to convert the given demand volumes to flow rates in pc/h. The PHF for both the ramp and freeway is 0.92 (given). Level terrain prevails: From Chapter 28, E_{HV} for level terrain is 2.0. Then:

$$f_{HV} = \frac{1}{1 + P_{HV}(E_{HV} - 1)}$$

$$f_{HV,\,freeway} = \frac{1}{1 + 0.08(2-1)} = 0.926$$

$$f_{HV,ramp} = \frac{1}{1 + 0.05(2-1)} = 0.952$$

Converting the demand volumes:

$$v = \frac{V}{PHF * f_{HV}}$$

$$v_F = \frac{4200}{0.92 * 0.926} = 4{,}930 \;\; pc/h$$

$$v_{R1} = \frac{700}{0.92 * 0.952} = 799 \;\; pc/h$$

$$v_{R2} = \frac{1000}{0.92 * 0.952} = 1{,}142 \;\; pc/h$$

Step 2: Determine the Flow in Lanes 1 and 2 Immediately Upstream of the Ramp
This is a simple isolated on-ramp on a 6-lane freeway. From Table 30-3, Equation 30-5 (from Table 30-2) is used to estimate P_{FM}.

$$P_{FM} = 0.5775 - 0.000028 L_a = 0.5775 - (0.000028 * 1000) = 0.5495$$

Note that the value of P_{FM} *does not* depend on the ramp flow rate, so it is the same for both demand scenarios presented. Then, using Equation 30-2:

$$v_{12} = v_F * P_{FM}$$
$$v_{12} = 4930 * 0.5495 = 2{,}709 \ pc/h$$

Likewise, this value does not change with the ramp flow rate.

Step 3: Check Capacity Values
Capacity values are given in Table 30-5. The critical capacity check is the flow rate on the freeway downstream of the on-ramp. The table below illustrates the capacity checks for this problem.

Capacity Checks for Problem 30-2

Component	Demand Flow Rate (pc/h)	Capacity (pc/h)	OK?
Freeway (v_F + v_R) *Scenario 1*	4930+799 = 5,729 pc/h	7,050 (Table 28-5, 3 lanes, FFS = 65)	Yes
Ramp (v_R) *Scenario 1*	799 pc/h	2,000 pc/h (Table 28-5, RFFS = 40)	Yes
Flow Entering Influence Area (v_{R12}) *Scenario 1*	2709+799 = 3,508	4,600 (Table 28-5)	Yes
Freeway (v_F + v_R) *Scenario 2*	4930+1152 = 6,072	7,050 (Table 28-5, 3 lanes, FFS = 65)	Yes
Ramp (v_R) *Scenario 2*	1152	2,000 pc/h	Yes
Flow Entering Influence Area (v_{R12}) *Scenario 2*	2709+1152 = 3,861	4,600 pc/h	Yes

All of the capacity checks for both demand scenarios, and no failures are expected. The analysis may continue.

Step 4: Determine Density and LOS
Density for an on-ramp is estimated using Equation 30-21:

$$D_R = 5.475 + 0.00734 v_R + 0.0078 v_{12} - 0.00627 L_a$$
$$D_{R,Scenario1} = 5.475 + (0.00734 * 799) + (0.0078 * 2709) - (0.00627 * 1000)$$
$$D_{R,Scenario1} = 5.475 + 5.865 + 21.130 - 6.270 = 26.2 \ pc/mi/\ln$$
$$D_{R,Scenario2} = 5.475 + (0.00734 * 1142) + (0.0078 * 2709) - (0.00627 * 1000)$$
$$D_{R,Scenario2} = 5.475 + 8.382 + 21.130 - 6.270 = 28.7 \ pc/h$$

From Table 30-1, the LOS for Scenario 1 is C, while for Scenario 2, it is D – just over the maximum for LOS C, which is 28 pc/mi/ln.

Step 5: Determine Speed Behavior

Table 30-6 gives equations for the determination of average speed in a merge segment. The following equations are used:

$$M_S = 0.321 + 0.0039e^{(v_{R12}/1000)} - 0.002(L_a * RFFS/1000)$$
$$S_R = FFS * SAF - (FFS * SAF - 42)M_S$$
$$S_o = FFS * SAF - 0.0036(v_{oa} - 500)$$
$$S = \frac{v_{R12} + v_{oa}\,N_o}{\left(\frac{v_{R12}}{S_R}\right) + \left(\frac{v_{oa}N}{S_o}\right)}$$

These speeds are computed as shown, and displayed in the table below.

Speed Results for Problem 30-2

Scenario	Average Speed in Ramp Influence Area, S_R	Average Speed in Outer Lane, S_o	Average of All Speeds, S
Scenario 1	56.4 mi/h	58.8 mi/h	57.3 mi/h
Scenario 2	55.2 mi/h	58.8 mi/h	56.5 mi/h

Analysis

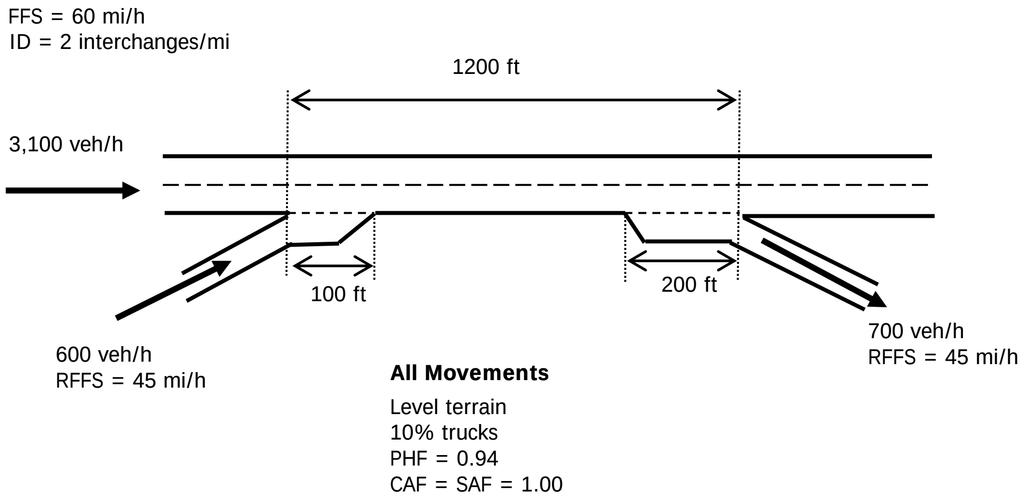
In this case, there is very little impact of changing the on-ramp demand volume from 700 veh/h to 1,000 veh/h. While the LOS technically changes from C to D, the change is small in density, but happens to straddle the boundary between the two. The change in LOS looks larger than it really is. Speed behavior also does not change very much, although there is minor slowing with the additional 300 veh/h added to the demand.

Solution to Problem 30-3

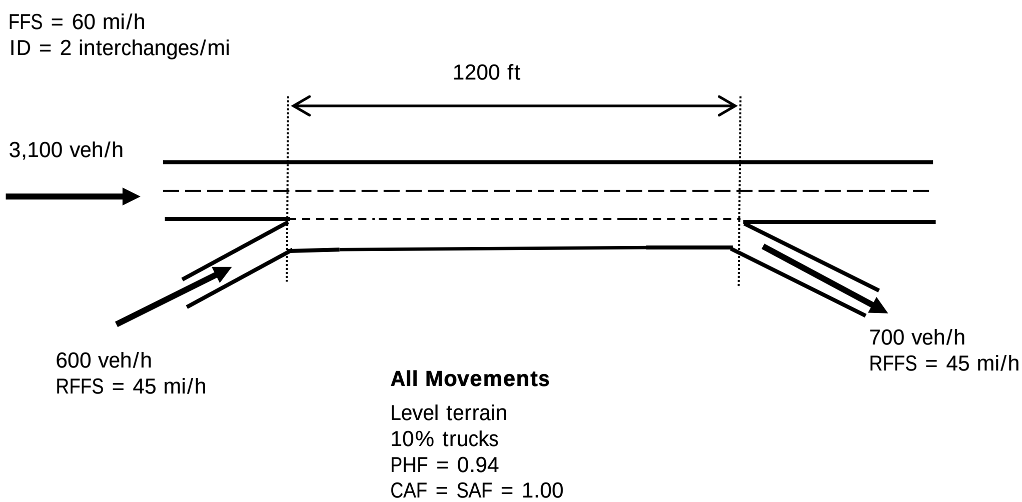
This problem actually requires four separate analyses, because it asks the analyst to evaluate existing operations, and then projects three different improvement scenarios. The four scenarios are shown below.

Scenario 1: Existing Case

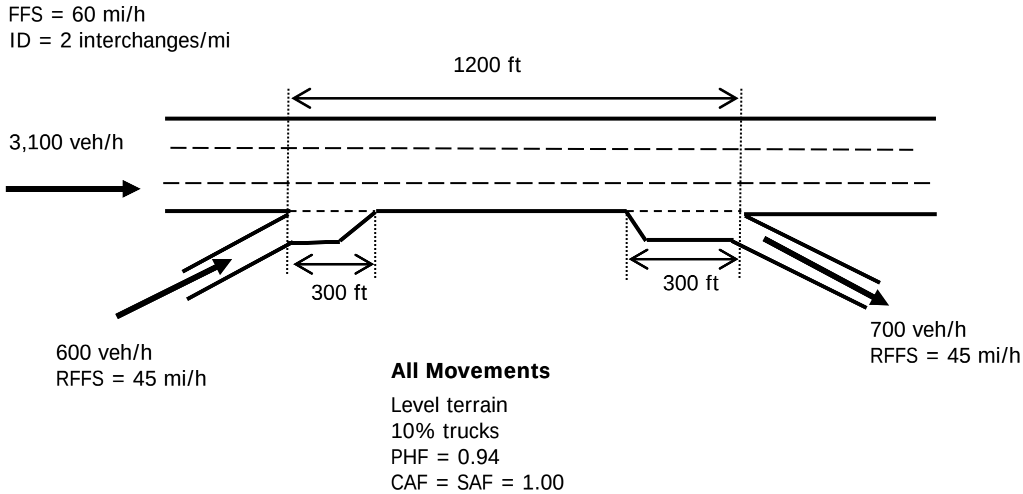
Scenario 1 is the existing situation with consecutive on- and off-ramps on a 4-lane freeway, shown below:



Scenario 2: Add a Continuous Auxiliary Lane Between the Ramps
In this scenario, the addition of an auxiliary lane creates a ramp-weave segment, which must be analyzed using the methodology of Chapter 29.

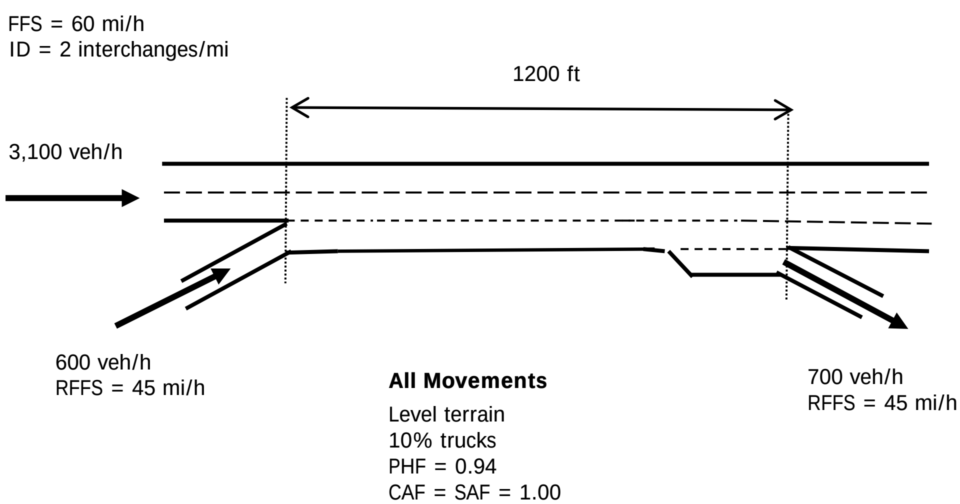


Scenario 3: Add a Third Freeway Lane, Extend Acceleration and Deceleration Lanes
In this scenario, a third freeway lane is added, creating a 6-lane freeway, and the acceleration and deceleration lanes are extended to 300 ft each.



Scenario 4: Provide a Lane Addition at the On-Ramp

In this scenario, a lane is added at the on-ramp and carries through past the off-ramp. This also creates a weaving segment, and would be analyzed using the methodology of Chapter 29.



Each of these scenarios is analyzed in the sections that follow. In some cases, not all of the details of computations are shown, to save space. All values are summarized, however, for examination and discussion.

All computations have been demonstrated in Chapter 29 and 30 example problems, and in solutions to previous homework questions.

Scenario 1: Existing Case

Step 1: Convert Demand Volumes to Flow Rates in pc/h

All demands have a PHF of 0.94 and include 10% trucks in level terrain. From Chapter 28, for level terrain, $E_{HV} = 2$. Then:

$$f_{HV} = \frac{1}{1+P_{HV}(E_{HV}-1)} = \frac{1}{1+0.10(2-1)} = 0.909$$

Demand volumes may now be converted using Equation 30-1:

$$v = \frac{V}{PHF * f_{HV}}$$
$$v_{F1} = \frac{3100}{0.94*0.909} = 3{,}628 \ pc/h$$
$$v_{R1} = \frac{600}{0.94*0.909} = 702 \ pc/h$$
$$v_{R2} = \frac{700}{0.94*0.909} = 819 \ pc/h$$

Note also that $v_{F2} = v_{F1}+v_{R1}$ = 3628+702 = 4,330 pc/h.

Step 2: Determine v_{12} Immediately Upstream of Each Ramp

In this case, this is a trivial task, as there are only two lanes in each direction on the 4-lane freeway. Thus, all demand is restricted to these lanes. Therefore:

- $v_{12(1)} = v_{F1}$ = 3,628 pc/h
- $v_{12(2)} = v_{F2}$ = 4,330 pc/h

Step 3: Capacity Checks

All capacity values are found in Table 30-5. The critical checkpoints are the freeway flow between the two ramps (v_{F2}), the two ramp flows (v_{R1} and v_{R2}), and flow entering the ramp influence areas ($v_{R12(1)}$ and $v_{12(2)}$). These checks are carried out in the table below.

Capacity Checks for Problem 30-3, Scenario 1

Element	Demand Flow Rate	Capacity	OK?
Freeway Flow Rate, v_{F2}	4,330 pc/h	4,600 pc/h (Table 30-5, 2 lanes, FFS = 60)	Yes
Ramp 1, v_{R1}	702 pc/h	2,100 pc/h (Table 30-5, one lane, RFFS = 45)	Yes
Ramp 2, v_{R2}	819 pc/h	2,100 pc/h (Table 30-5, one lane, RFFS = 45)	Yes
Flow Entering On-Ramp Influence Area, $v_{R12(1)}$	3628+702=4,330 pc/h	4,600 pc/h (Table 30-5)	Yes
Flow Entering the Off-Ramp Influence Area, $v_{12(2)}$	4,330 pc/h	4,400 pc/h (Table 30-5)	Yes

Step 4: Estimate Density and LOS

Density in the merge segment (Ramp 1) is estimated using Equation 30-21. Density in the diverge segment (Ramp 2) is estimated using Equation 30-22.

$$D_{R1} = 5.475 + 0.00734\,v_{R1} + 0.0078\,v_{12(1)} - 0.00627\,L_a$$
$$D_{R1} = 5.475 + (0.00734 * 702) + (0.0078 * 3628) - (0.00627 * 100)$$
$$D_{R1} = 5.475 + 5.153 + 28.298 - 0.627 = 38.3\ pc/mi/\ln\ (LOS\ E, Table\ 30-1)$$
$$D_{R2} = 4.252 + 0.0086\,v_{12(2)} - 0.009\,L_d$$
$$D_{R2} = 4.252 + (0.0086 * 4330) - (0.009 * 200)$$
$$D_{R2} = 4.252 + 37.238 - 1.800 = 39.7\ pc/h/\ln\ (LOS\ E,\ Table\ 30-1)$$

The segment is expected to operate very poorly, in LOS E for both the on-ramp and off-ramp influence areas (which covers the entire freeway in this case).

Step 5: Estimate Speed Behavior

Equations for the estimation of speed in the merge influence area are given in Table 30-6, while equations for the diverge influence area are given in Table 30-7. The details of speed computations are not shown, but the results are summarized below. Note that there are no “outer lanes” in this case, and the speed within the ramp influence areas are the same as the speed of all vehicles.

- $S_{R1} = S_1 = 52.6$ mi/h
- $S_{R2} = S_2 = 49.2$ mi/h

Speeds are relatively low, as expected. Because the distance between the ramps is only 1,200 ft, the two 1,500 ft influence areas extend across this entire length, and the poorer operation prevails. Thus, the expected average speed between the ramps is expected to be 49.2 mi/h, with LOS E conditions prevailing.

Scenario 2: Connect the Ramps with a Continuous Auxiliary Lane

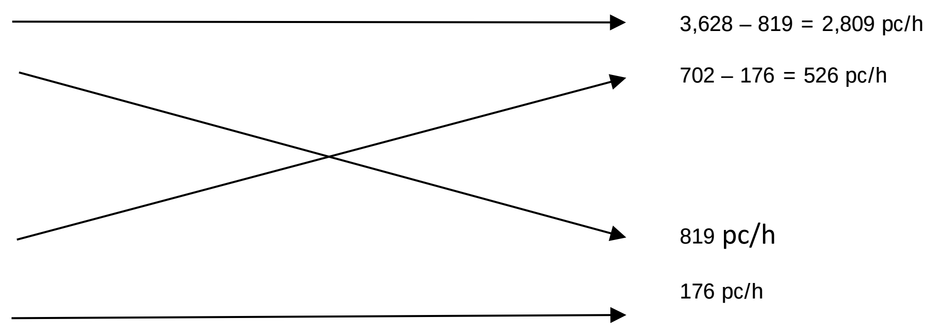
In this case, a ramp-weave segment with three total lanes (including the auxiliary lane) is created. Because it is a ramp weave, the following key configuration characteristics are known (see Chapter 29):

- $LC_{RF} = 1$
- $LC_{FR} = 1$
- $N_{WV} = 2$

It is also necessary to create a weaving diagram. The basic flows have already been converted to pc/h under Scenario 1, and would be applied here. The problem statement, however, indicates that there are 150 veh/h travelling from ramp to ramp. This would also have to be converted to a flow rate in pc/h, using the same PHF and f_{HV} used in Scenario 1. Thus:

$$v_{RR} = \frac{150}{0.94 * 0.909} = 176 \ pc/h$$

These 176 pc/h are *part of* the on-ramp flow rate. They become an outer flow in a weaving segment. Note also that the off-ramp flow enters the segment as *part of* the entering freeway flow. The weaving diagram for the segment may now be constructed:



Then:

v_{FF} = 2,809 pc/h
v_{FR} = 819 pc/h
v_{RF} = 526 pc/h
v_{RR} = 176 pc/h
v_W = 819 + 526 = 1,345 pc/h
v_{NW} = 2,809 + 176 = 2,985 pc/h
v = 1,345 + 2,985 = 4,330 pc/h
VR = 1345/4330 = 0.3106

The length of the segment between ramps is given as 1,200 ft. For the purposes of this computation, it will be assumed that there are no barrier markings restricting lane-changing within the segment. Thus, L_S = 1,200 ft.

Determining Configuration Characteristics

Several key characteristics were discussed previously: LC_{FR}, LC_{RF}, and N_{WV}. The value of LC_{MIN} for a one-sided weaving segment is estimated using Equation 29-2:

$$LC_{MIN} = (LC_{FR} * v_{FR}) + (LC_{RF} * v_{RF}) = (1*819) + (1*526) = 1{,}345 \ lc/h$$

Determine the Maximum Weaving Length

The maximum weaving length is estimated using Equation 29-4:

$$L_{MAX} = \left[5{,}728(1+VR)^{1.6}\right] - \left[1{,}566\, N_{WV}\right]$$
$$L_{MAX} = \left[5{,}728 * 1.3106^{1.6}\right] - \left[1{,}566 * 2\right] = 8{,}830 - 2{,}132 = 6{,}698\ ft$$

As the actual length of the weaving segment is less than the maximum, the segment *is* operating as a weaving segment, and the analysis continues.

<u>Determine the Capacity of the Weaving Segment</u>
The capacity of a weaving segment may be estimated in two ways. The first is based upon capacity occurring when the density reaches 43 pc/mi/ln, and is estimated using Equations 29-5 and 29-6:

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765\, L_S\right] + \left[119.8\, N_{WV}\right]$$

The c_{IFL} for a freeway with a 60-mi/h FFS is 2,300 pc/h/ln (Table 29-2). Then:

$$c_{IWL} = 2{,}300 - \left[438.2\left(1.3106^{1.6}\right)\right] + \left[0.0765 * 1{,}200\right] + \left[119.8 * 2\right]$$
$$c_{IWL} = 2{,}300 - 675.5 + 91.8 + 239.6 = 1{,}956\ \ pc/h/\ln$$

Since there are three lanes in the segment, the capacity of the segment is:

$$c_{IW} = 1956 * 3 = 5{,}868\ \ pc/h$$

The second limitation on capacity is the maximum weaving flow. This capacity is estimated using Equation 29-7:

$$c_{IW} = 2400/VR = 2400/0.3106 = 7{,}727\ \ pc/h$$

The lower value, 5,868 pc/h prevails. As this is larger than the total flow rate in the segment (4,330 pc/h), no failure is expected, and the analysis continues.

<u>Determine Lane-Changing Rates for Weaving and Non-Weaving Vehicles</u>
The lane-changing rat for weaving vehicles is estimated using Equation 29-12:

$$LC_W = LC_{MIN} + 0.39\left[(L_S - 300)^{0.5}\, N^2\, (1+ID)^{0.8}\right]$$
$$LC_W = 1{,}345 + 0.39\left[(1{,}200 - 300)^{0.5} * 3^2 * (1+2)^{0.8}\right]$$
$$LC_W = 1{,}345 + 254 = 1{,}599\ lc/h$$

The lane-changing rate for non-weaving vehicles is estimated using one of Equations 29-13. The choice depends upon the value of the non-weaving lane-changing index, computed as:

$$I_{NW} = \frac{L_S \, ID \, v_{NW}}{10{,}000} = \frac{1{,}200 * 2 * 2{,}986}{10{,}000} = 716.6 < 1{,}300$$ Because of this value, the *first* Equation 29-13 is used:

$$LC_{NW} = (0.206 v_{NW}) + (0.542 L_S) - (192.6 N)$$
$$LC_{NW} = (0.206 * 2{,}986) + (0.542 * 1{,}200) - (192.6 * 3)$$
$$LC_{NW} = 615.2 + 650.4 - 577.8 = 688 \; lc/h$$

The total lane-changing rate is the sum of:

$$LC_{ALL} = LC_W + LC_{NW} = 1{,}599 + 688 = 2{,}287 \; lc/h$$

Estimate Speeds in the Weaving Segment

The average speed of weaving vehicles in the weaving segment is given by Equations 29-18 and 29-19:

$$W = 0.226\left(\frac{LC_{ALL}}{L_S}\right)^{0.789} = 0.226\left(\frac{2{,}287}{1{,}200}\right)^{0.789} = 0.3759$$
$$S_W = 15 + \left(\frac{FFS * SAF - 15}{1 + W}\right) = 15 + \left(\frac{60*1 - 15}{1 + 0.3759}\right) = 47.7 \; mi/h$$

The average speed of non-weaving vehicles is estimated using Equation 29-20:

$$S_{NW} = FFS * SAF - (0.0072 LC_{MIN}) - 0.0048\left(v/N\right)$$
$$S_{NW} = 60*1 - (0.0072 * 1{,}345) - (0.0048 * 4{,}330/3)$$
$$S_{NW} = 60.0 - 9.7 - 6.9 = 43.4 \; mi/h$$

The average speed of all vehicles is computed using Equation 29-21:

$$S = \frac{v_W + v_{NW}}{\left(\frac{v_W}{S_W}\right) + \left(\frac{v_{NW}}{S_{NW}}\right)} = \frac{1{,}345 + 2{,}985}{\left(\frac{1345}{47.7}\right) + \left(\frac{2985}{43.4}\right)} = \frac{4330}{28.20 + 68.78} = \frac{4330}{96.98} = 44.6 \; mi/h$$

Estimate the Density and LOS of the Weaving Segment

The density of the segment is estimated using Equation 29-22:

$$D = \frac{v/N}{S} = \frac{4330/3}{44.6} = 32.4 \; pc/h/\text{ln}$$

From Table 29-1, this represents LOS D. The result is a bit better than the LOS E that resulted from the original configuration. Speeds, however, are a bit slower in the weaving segment than the original ramp sequence. The greater spatial distribution of lane-changing in the weaving configuration may account for this.

Scenario 3: Add a Third Freeway Lane, Increase Accel-Decel Lanes to 300ft
This creates another ramp sequence with the same basic dimensions as Scenario 1. The freeway is now a base 6-lane freeway, which will change the computations for v_{12}, and any subsequent computations depending on those results.

All volumes have already been converted to flow rates in pc/h. They are repeated here for convenience:

v_{F1} = 3,628 pc/h
v_{F2} = 4,330 pc/h
v_{R1} = 702 pc/h
v_{R2} = 819 pc/h

<u>Recompute Flow Rates in Lanes 1 and 2 Immediately Upstream of the Ramps</u>
The on-ramp (Ramp 1) has a downstream off-ramp. From Table 30-3, either Equation 30-5 or 30-7 could apply. The decision is based on the equivalence distance, L_{EQ}, which is computed using Equation 30-16 in Table 30-4:

$$L_{EQ} = \frac{v_d}{0.1096 + 0.000107\,L_d} = \frac{819}{0.1096 + (0.000197 * 300)} = 4{,}855\ ft > 1{,}200\,ft$$

As the actual downstream ramp falls within the influence range, the equation that addresses the interaction – Equation 30-7 – is used to find v_{12}:

$$P_{FM} = 0.5487 + 0.2628\left(\frac{v_d}{L_{DN}}\right) = 0.5487 + 0.2628\left(\frac{819}{1200}\right) = 0.7281$$
$$v_{12(1)} = v_{F1}\,P_{FM} = 3{,}628 * 0.7281 = 2{,}642\ \ pc/h$$

This must be checked for reasonableness. The outer lane flow is 3,628-2,642 = 986 pc/h, which is less than 2,700 pc/h. Further, it is less than 1.5 (2642/2) = 1,982 pc/h. The distribution is, therefore, reasonable, and the results will hold.

The off-ramp (Ramp 2) has an upstream on-ramp. From Table 30-3, either Equation 30-11 or 30-12 might apply. Again, an equivalence distance must be computed and compared to the actual distance between ramps. Equation 30-17 is used to do this:

$$L_{EQ} = \frac{v_U}{0.071 + 0.000023 v_{F2} - 0.000076 v_{R2}}$$
$$\frac{702}{0.071 + (0.000023 * 4330) - (0.000076 * 819)} = 36{,}947\ ft > 1{,}200\ ft$$

Again, the ramp falls within the influence distance, and the equation considering the interaction would be used – Equation 30-12:

$$P_{FD} = 0.717 - 0.000039 v_{F2} + 0.604\left(v_U / L_{UP} \right)$$
$$P_{FD} = 0.717 - (0.000039 * 4330) + (0.604 * 702/1200) = 0.901$$
$$v_{12(2)} = v_{R2} + (v_{F2} - v_{R2}) * 0.901 = 819 + (4330 - 819) * 0.901 = 3{,}982 \ pc/h$$

Once again, this result must be checked for reasonableness. The flow in the outer lane is only 4,330-3,982 = 348 pc/h. This is less than 2,700 pc/h, and less than 1.5 (3982/2) = 2,987 pc/h. Thus, the result may be taken as reasonable. A word of caution is advised: The HCM does not mandate a "reasonableness" test for *underassigning* vehicles to outer lanes, which appears to be the case here. The analysis continues, however, with the realization that the resulting LOS is a worst case that might be somewhat better with more vehicles using the outer lane.

Check Capacity Values

Capacity values are given in Table 30-5. Comparisons to demand flow rates are summarized in the table below.

Capacity Checks for Problem 30-3, Scenario 3

Element	Demand Flow Rate	Capacity	OK?
Freeway Flow Rate, v_{F2}	4,330 pc/h	4,600 pc/h (Table 30-5, 2 lanes, FFS = 60)	Yes
Ramp 1, v_{R1}	702 pc/h	2,100 pc/h (Table 30-5, one lane, RFFS = 45)	Yes
Ramp 2, v_{R2}	819 pc/h	2,100 pc/h (Table 30-5, one lane, RFFS = 45)	Yes
Flow Entering On-Ramp Influence Area, $v_{R12(1)}$	2642+702=3,344 pc/h	4,600 pc/h (Table 30-5)	Yes
Flow Entering the Off-Ramp Influence Area, $v_{12(2)}$	3,982 pc/h	4,400 pc/h (Table 30-5)	Yes

Determine the Density LOS for the Ramp Influence Areas

The density in the Ramp 1 influence area is estimated using Equation 30-21:

$$D_{R1} = 5.475 + 0.00734 v_{R1} + 0.0078 v_{12(1)} - 0.00627 L_a$$
$$D_{R1} = 5.475 + (0.00734 * 702) + (0.0078 * 2642) - (0.00627 * 300)$$
$$D_{R1} = 5.475 + 5.153 + 20.608 - 1.881 = 29.4 \ pc/h/\ln \ \ (LOS\ D, Table\ 30-1)$$

The density in the Ramp 2 influence areas is estimated using Equation 30-22:

$$D_{R2} = 4.252 + 0.0086 v_{12(2)} - 0.009 L_d$$
$$D_{R2} = 4.252 + (0.0086 * 3982) - (0.009 * 300) = 35.8 \ pc/mi/\ln \ \ (LOS\ E, Table\ 30-1)$$

This still results in a LOS of E, the same as in the case of the 4-lane freeway, even though the density is a bit lower. Because the influence areas fully overlap, LOS E prevails throughout the segment.

Estimate Speed Behavior
Speed computations are not shown, but result are summarized below:

Speed Results for Problem 30-3, Scenario 3

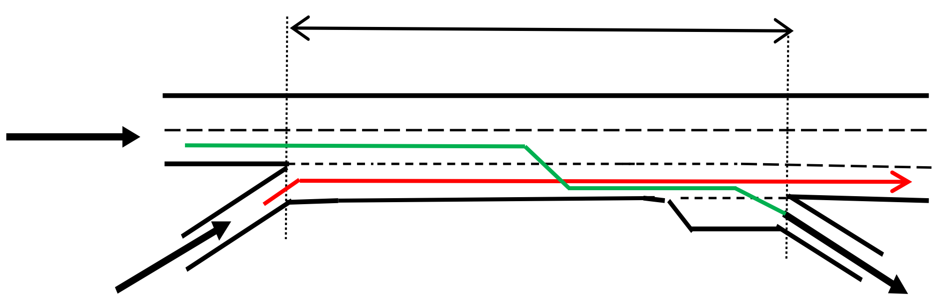
Element	Ramp Influence Area	Outer Lane	All Lanes
Ramp 1	56.9 mi/h	63.3 mi/h	58.2 mi/h
Ramp 2	49.9 mi/h	65.5 mi/h	50.9 mi/h

Again, as the ramp influence areas completely overlap, the Ramp 2 speed (all lanes) of 50.9 mi/h controls the entire segment.

Scenario 4: Lane Addition at On-Ramp
The addition of a lane at the on-ramp creates a 6-lane freeway at that point. It also creates a weaving segment of 3 total lanes – similar to Scenario 2. The difference, however, is in the configuration of the weaving segment.

The configuration is neither a pure ramp-weave or a pure major weave. There is no "auxiliary lane," as the added lane continues after the second ramp. There are only two legs with more than one lane. Neither strict definition applies.

The basic characteristics of the weaving configuration must still be determined, however, to proceed.



As seen in the illustration, $LC_{RF} = 0$, while $LC_{FR} = 1$. In the latter case, it is important to note that the existence of the short deceleration lane does not count as an additional lane change from the point of view of weaving analysis. In fact, as a weaving analysis, the deceleration lane will not even enter the computations. Lastly, there are only two lanes from which a weave can be made with one or fewer lane changes, thus $N_{WV} = 2$.

All of the computations from Scenario 2 are repeated. The key change is in lane-changing, which significantly reduced in this configuration compared to the first. Note that the weaving diagram for Scenario 2 does not change, and applies here as well.

Determining Configuration Characteristics

Several key characteristics have been determined: LC_{FR} =1, LC_{RF} =0, and N_{WV} = 2. The value of LC_{MIN} for a one-sided weaving segment is estimated using Equation 29-2:

$$LC_{MIN} = (LC_{FR} * v_{FR}) + (LC_{RF} * v_{RF}) = (1*819) + (0*526) = 819\ lc/h$$

Determine the Maximum Weaving Length

The maximum weaving length is estimated using Equation 29-4:

$$L_{MAX} = \left[5{,}728(1+VR)^{1.6}\right] - \left[1{,}566\,N_{WV}\right]$$
$$L_{MAX} = \left[5{,}728*1.3106^{1.6}\right] - \left[1{,}566*2\right] = 8{,}830 - 2{,}132 = 6{,}698\ ft$$

As the actual length of the weaving segment is less than the maximum, the segment *is* operating as a weaving segment, and the analysis continues.

Determine the Capacity of the Weaving Segment

The capacity of a weaving segment may be estimated in two ways. The first is based upon capacity occurring when the density reaches 43 pc/mi/ln, and is estimated using Equations 29-5 and 29-6:

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765\,L_S\right] + \left[119.8\,N_{WV}\right]$$

The c_{IFL} for a freeway with a 60-mi/h FFS is 2,300 pc/h/ln (Table 29-2). Then:

$$c_{IWL} = 2{,}300 - \left[438.2\left(1.3106^{1.6}\right)\right] + \left[0.0765*1{,}200\right] + \left[119.8*2\right]$$
$$c_{IWL} = 2{,}300 - 675.5 + 91.8 + 239.6 = 1{,}956\ pc/h/\ln$$

Since there are three lanes in the segment, the capacity of the segment is:

$$c_{IW} = 1956*3 = 5{,}868\ pc/h$$

The second limitation on capacity is the maximum weaving flow. This capacity is estimated using Equation 29-7:

$$c_{IW} = 2400/VR = 2400/0.3106 = 7{,}727\ pc/h$$

The lower value, 5,868 pc/h prevails. As this is larger than the total flow rate in the segment (4,330 pc/h), no failure is expected, and the analysis continues.

Determine Lane-Changing Rates for Weaving and Non-Weaving Vehicles
The lane-changing rat for weaving vehicles is estimated using Equation 29-12:

$$LC_W = LC_{MIN} + 0.39\left[(L_S - 300)^{0.5} N^2 (1 + ID)^{0.8}\right]$$
$$LC_W = 819 + 0.39\left[(1{,}200 - 300)^{0.5} * 3^2 * (1 + 2)^{0.8}\right]$$
$$LC_W = 819 + 254 = 1{,}073\ lc/h$$

The lane-changing rate for non-weaving vehicles is estimated using one of Equations 29-13. The choice depends upon the value of the non-weaving lane-changing index, computed as:

$$I_{NW} = \frac{L_S\, ID\, v_{NW}}{10{,}000} = \frac{1{,}200 * 2 * 2{,}986}{10{,}000} = 716.6 < 1{,}300$$

Because of this value, the *first* Equation 29-13 is used:

$$LC_{NW} = (0.206\, v_{NW}) + (0.542\, L_S) - (192.6\, N)$$
$$LC_{NW} = (0.206 * 2{,}986) + (0.542 * 1{,}200) - (192.6 * 3)$$
$$LC_{NW} = 615.2 + 650.4 - 577.8 = 688\ lc/h$$

The total lane-changing rate is the sum of:

$$LC_{ALL} = LC_W + LC_{NW} = 1{,}073 + 688 = 1{,}761\ lc/h$$

Estimate Speeds in the Weaving Segment
The average speed of weaving vehicles in the weaving segment is given by Equations 29-18 and 29-19:

$$W = 0.226\left(\frac{LC_{ALL}}{L_S}\right)^{0.789} = 0.226\left(\frac{1{,}761}{1{,}200}\right)^{0.789} = 0.3059$$
$$S_W = 15 + \left(\frac{FFS * SAF - 15}{1 + W}\right) = 15 + \left(\frac{60 * 1 - 15}{1 + 0.3059}\right) = 49.5\ mi/h$$

The average speed of non-weaving vehicles is estimated using Equation 29-20:

$$S_{NW} = FFS * SAF - (0.0072\, LC_{MIN}) - 0.0048\left(v/N\right)$$
$$S_{NW} = 60 * 1 - (0.0072 * 819) - (0.0048 * 4{,}330/3)$$
$$S_{NW} = 60.0 - 5.9 - 6.9 = 47.2\ mi/h$$

The average speed of all vehicles is computed using Equation 29-21:

$$S = \frac{v_W + v_{NW}}{\left(\frac{v_W}{S_W}\right) + \left(\frac{v_{NW}}{S_{NW}}\right)} = \frac{1{,}345 + 2{,}985}{\left(\frac{1345}{49.5}\right) + \left(\frac{2985}{47.2}\right)} = \frac{4330}{27.17 + 63.25} = \frac{4330}{90.42} = 47.9\ mi/h$$

<u>Estimate the Density and LOS of the Weaving Segment</u>

The density of the segment is estimated using Equation 29-22:

$$D = \frac{v/N}{S} = \frac{4330/3}{47.9} = 30.2\ pc/h/\ln$$

From Table 29-1, this still represents LOS D.

<u>Comparison of Scenarios</u>

Fundamental results of the four different alternatives are summarized in the table below.

Comparison of Scenarios

Scenario	S_R	S_O	S	D	LOS
Existing Condition – 1	49.2 mi/h	-	49.2 mi/h	39.7 pc/mi/ln	E
Ramp-Weave – 2	-	-	44.6 mi/h	32.4 pc/mi/ln	D
Add Freeway Lane – 3	49.9 mi/h	65.5 mi/h	50.9 mi/h	35.8 pc/mi/ln	E
Add Lane at Ramp – 4	-	-	47.9 mi/h	30.2 pc/mi/ln	D

The most obvious conclusion is that both weaving configurations produce a better LOS than the ramp sequences. Looking more closely at the figures, however, the improvement is not great. Densities are lower for the weaving segments, but speeds are also slower. This may have more to do with the models for prediction, particularly the predictions of outer lane speed for ramp configurations. In this case, the assignment of flow to the outer lane of a ramp sequence seems too low for Scenario 3, which contributes to a higher overall speed prediction.

In the final analysis, none of these scenarios will work "well." Some congestion will be present for any of them. Overall, Scenario 4 seems to produce the lowest densities, and the speeds are reasonable.

The density comparisons are also difficult, as the ramp methodology produces a density in the ramp influence area (lanes 1 and 2), whereas the weaving methodology produces a density across all freeway lanes. The outer lane density in Scenario 3, for example, would be quite low, given the small assignment of demand to that lane.

Comparing across different HCM methodologies is often difficult, as they were calibrated at different times, and use different fundamental approaches.

Solutions to Problems in Chapter 31

Operation of Freeways and Highways

Problem 31-1
The length of the taper transition, with a speed limit of 55 mi/h, is given by Equation 31-1:

$$L = WS$$

where: W = width of the middle lane (assumed to be 12 ft)
S = speed limit (55 mi/h)

$$L = 12 * 55 = 660\ ft$$

From Figure 31-4, the length of the buffer zone must be *at least* the sight distance in either direction, and never less than 50 ft. From Table 31-1, for 55 mi/h, the passing sight distance is 900 ft. Thus, the minimum length of the buffer zone is 900 ft.

Problem 31-2
There are two issues:

- The speed limit should *never* be more than the design speed of 65 mi/h.
- The 85th percentile speed is 7 mi/h over the design speed.

As the existing speed limit is not stated, it is impossible to evaluate its impact on the situation.

If the current speed limit is more than 65 mi/h, it should immediately be lowered to this value. If the current speed limit is 65 mi/h or less, stepped up enforcement must be considered. Lowering the speed limit to below 65 mi/h would probably be ineffective since drivers are already driving well over that speed.

Problem 31-3
This is a class project that will differ for each student team, depending upon the section of freeway they select for study.

Problem 31-4

- In each direction, advance interchange signs should be placed at 10 miles, 5 miles, 2 miles, 1 mile – and perhaps at ½ mile from each exit ramp. An advance sign at 20 miles would also be considered.

- At the diverge points (in both directions), a cantilevered overhead sign showing the exit number (if one exists), the interchange with County Road 17, the destinations in each direction of County Road 17, and an appropriate arrow showing the exit.

- At the far side of each intersection (from the off ramp), a destination sign with arrows showing which direction to turn for which destination(s) would be placed.

- On County Road 17, in both directions, advance guide signs showing the upcoming interchange with State Route 70, and destinations served.

- At the intersections, overhead signs should direct drivers into the appropriate lane(s) to access the direction of State Route 70 they desire.

Problem 31-5

There are three types of managed lanes in common use in the U.S.:

- High-Occupancy Vehicle (HOV) lanes: limited to car pools of x persons or more during designated hours of the day; buses and taxis may or may not be included.

- High-Occupancy Toll (HOT) lanes: limited to car pools, but permitting others to use the lane by paying a toll or fee; an appropriate readable tag is usually required for all vehicles to use the lane.

- Express Toll lanes: open to all vehicles by paying a toll or fee; an appropriate readable tag is usually required for all vehicles to use the lane.

Problem 31-6

Advanced Traffic and Demand Management Strategies (ATDM) involve a wide range of traffic engineering measures applied to improve the performance and/or capacity of major highway or street networks. These include:

- Managed lanes.
- Dynamic ramp metering.
- Dynamic lane use controls.
- Dynamic speed limits.
- Dynamic pricing strategies.
- Dynamic traveler information.

The key word in most of these is "dynamic." Sensors and cameras monitor all aspects of system operation, and controls are varied in real time to address identified congestion, incidents, or accidents.

Managed lanes attempt to lure travelers to form car pools to access higher-speed HOV, HOT, or express toll lanes. They also generate revenue to support the system. Pricing (tolls) are often varied by time of day, or on the basis of current operating conditions in the lane vs. general use lanes.

Ramp metering limits the number of vehicles that can enter the freeway at each on-ramp to maintain reasonable operating quality on the freeway. It must be done carefully, and in a way that does not cause bigger problems elsewhere in the system as vehicles are diverted from the freeway.

Dynamic speed limits attempt to smooth traffic flow by maintaining consistent operating speeds, and may be assigned by lane.

Dynamic lane use comes in many forms. The most common is allowing vehicles to use a paved shoulder as an active lane under certain conditions. Variable message lane use control signs must be frequently placed to implement this, or change the limitations on lane use for any lane or conditions.

Dynamic traveler information uses current information on conditions to direct travelers to the most efficient routes to their intended destination.

Obviously, these systems must all be carefully developed, and involve much variable signing, many sensors to detect current conditions, and software that uses sensor inputs to control the message displayed on each sign.

Problem 31-7

The free-flow travel time through the 15-mile segment of highway is 15 mi/70 mi/h = 0.214 hours or 0.214*60 = 12.86 minutes. The traveler reliability indices would then be:

$$TTI_{95} = 33.00/12.86 = 2.57$$
$$TTI_{80} = 27.00/12.86 = 2.10$$
$$TTI_{50} = 15.00/12.86 = 1.17$$

Let us assume that all of these travel times reflect the three evening peak hours of a typical weekday. The indices mean that for any random day, during the three hours included in the study:

- 50% of the time, the travel time is more than 15.0 min, or 1.17 times the free-flow travel time.

- 20% of the time, the travel time is more than 27.0 min, or 2.10 times the free-flow travel time.

- 5% of the time, the travel time is more than 33.0 min, or 2.57 times the free-flow travel time.

This would have to be compared to local expectations and criteria, as well as to other facilities in the region, to make any judgment on their acceptability.

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