When the left turns precede the through and right-turn movements in the phasing sequence for an approach, they are referred to as leading left turns. When the left turns follow the through and right-turn movements, they are referred to as lagging left turns. Although not shown in this figure, it is also possible for a movement to be protected for a period of time and then permitted for a period of time, or vice versa. This is most commonly seen with left-turn movements, and is referred to as protected plus permitted or permitted plus protected, depending on the sequence.

EXAMPLE 7.6

Refer to the intersection shown in Fig. 7.9. Use the cross product guideline to determine if protected left-turn phases should be provided for any of the approaches.

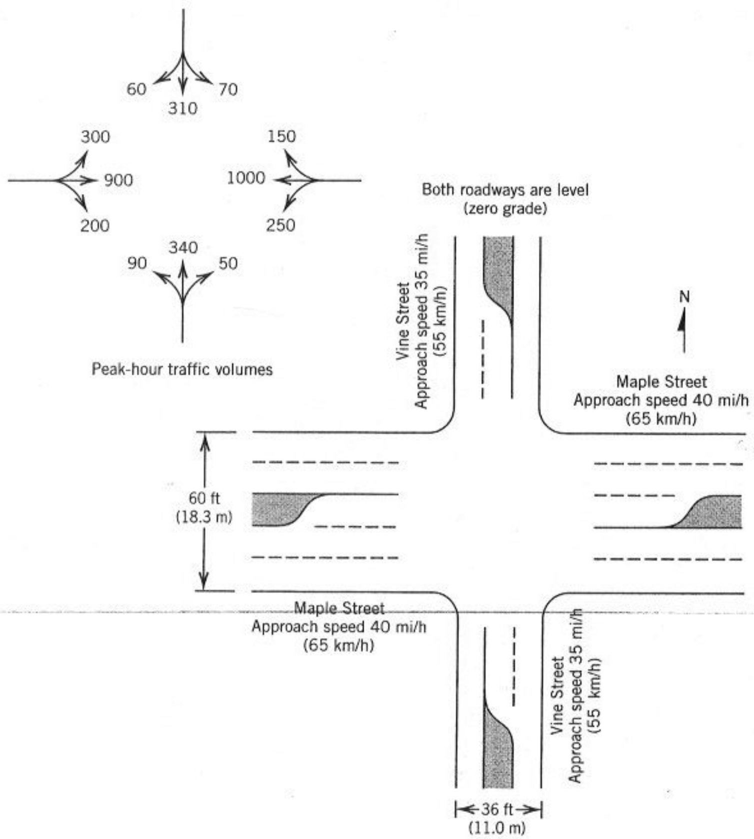

Figure 7.9 Intersection geometry and peak traffic volumes for example problems.

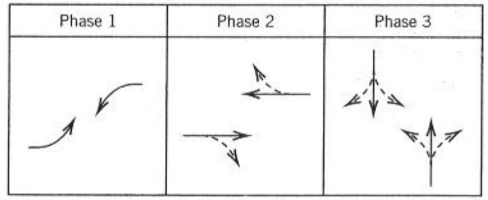

Figure 7.10 Recommended signal phasing plan for the intersection in Example 7.6.

SOLUTION

There are 250 westbound vehicles that turn left during the peak hour. The product of the westbound left-turning vehicles and the opposing eastbound traffic (right-turn and straight-through vehicles) is 275,000 [250 × (900 + 200)]. There are 300 eastbound vehicles that turn left during the peak hour. The product of the eastbound left-turning vehicles and the opposing westbound traffic (right-turn and straight-through vehicles) is 345,000 [300 × (1000 + 150)].

Because the cross product for each of these approaches is greater than 90,000 (the requirement for two opposing lanes), a protected left-turn phase is suggested for the WB and EB left turn movements. The NB and SB approaches do not require a protected left-turn phase using this criterion because the cross products for these approaches are less than 50,000 (for one opposing lane). Therefore, a three-phase traffic-signal plan is recommended, as shown in Fig. 7.10.

7.5.2 Establish Analysis Lane Groups

Each intersection approach is initially treated separately, and the results are later aggregated. Thus, each approach must be subdivided into logical groupings of traffic movements for analysis purposes. Based on the lane and traffic movement distribution on an approach, lane groups can be readily determined. The following general guidelines are offered for establishing lane groups [Transportation Research Board 2000].

- Movements made simultaneously from the same lane must be treated as a lane group.
- If an exclusive turn lane (or lanes) is present, it is usually treated as a separate lane group.
- If an approach includes an exclusive left-turn and/or right-turn lane, the remaining lanes are usually considered as a single lane group.

- If a multiple-lane approach includes a lane (or lanes) with shared movements, it must first be determined whether it really serves multiple movements or whether it is a de facto lane for one of the movements. For example, an approach that includes a shared left-turn and through movement lane may be operating primarily as a left-turn lane if another through lane is present. Likewise, it may be operating primarily as a through lane if an exclusive left-turn lane is also present.

Figure 7.11 shows some typical lane groupings for analysis purposes. Note that when multiple lanes are combined into a lane group, the subsequent analysis calculations for this lane group should treat these lanes as a single unit.

Number of lanes	Movements by lane	Number of possible lane groups
1	LT + TH + RT	① (Single-lane approach)
2	EXC LT TH + RT	②
2	LT + TH TH + RT	① or ②
3	EXC LT TH TH + RT	② or ③

Figure 7.11 Typical lane groupings for analysis (LT = left turn; TH = through; RT = right turn; EXC = exclusive).
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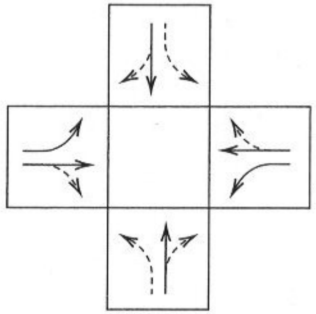

Figure 7.12 Analysis lane groups for the three-phase design at the intersection of Maple Street and Vine Street.

EXAMPLE 7.7

Determine the lane groups to use for analysis of the Maple and Vine streets intersection.

SOLUTION

The EB and WB left-turn movements will each be a lane group because they have a separate lane and move in a separate phase from the through/right-turn movements. Likewise, the EB and WB through/right-turn movements proceed together in a separate phase, and will therefore be separate lane groups. Although the right turns use only the outside lane, this movement's impact on the saturation flow rate for the two lanes combined will be determined. The NB and SB left turns will also each be a separate lane group. Even though they move during the same phase as the adjacent through and right-turn movements, these left turns are permitted and will have very different operating characteristics from the through and right-turn movements. Because the through and right-turn movements use the same lane, they will be an individual lane group for both the NB and SB approaches. The recommended lane groups for analysis for each of the approaches are shown in Fig. 7.12.

7.5.3 Calculate Analysis Flow Rates and Adjusted Saturation Flow Rates

Just as for the analysis of uninterrupted flow, the hourly traffic volume arriving on each intersection approach must be converted to an analysis flow rate that accounts for the peak 15-minute flow within that hour (typically the peak hour). This is accomplished by calculating the peak-hour factor (*PHF*) and dividing this into the hourly volume (as shown in Chapter 6), which yields the analysis flow rate.

One note about adjusting for the *PHF*. With the multiple traffic streams entering an intersection, a separate *PHF* can be calculated for each approach's traffic stream.

However, adjusting each approach volume by its specific *PHF* can yield unrealistically high combined analysis volumes, because the different approach volumes usually do not peak during the same 15-minute period. Applying a single *PHF* determined for the intersection as a whole will result in more reasonable analysis volumes.

The adjustment of the saturation flow rate was discussed in Section 7.3.1. Again, it is assumed that the approach volumes and saturation flow rates provided in this chapter have already been adjusted.

7.5.4 Determine Critical Lane Groups and Total Cycle Lost Time

For any combination of lane group movements during a particular phase, one of these lane groups will control the necessary green time for that phase. This lane group is referred to as the critical lane group. When the traffic movements of each lane group occur during only one phase of the signal cycle, the determination of the critical lane group for each phase is straightforward. In this case, the critical lane group for each phase is simply the lane group with the highest ratio of vehicle arrival rate to vehicle departure rate (λ/μ). This quantity is referred to as the flow ratio (which was called the traffic intensity, ρ, in Chapter 5) and is designated v/s (arrival flow rate divided by saturation flow rate). If the allocation of green time for each phase is based on the flow ratio of the critical lane group, then the noncritical lane group movements will be accommodated as well.

As previously discussed, with dual-ring controllers, a wide variety of phasing sequences are possible. The situation where a movement starts in one signal phase and continues in the next signal phase is referred to as an overlapping phase (see Fig. 7.8). For the case of overlapping phases in a signal cycle, the identification of the critical lane groups is more complex, as the lane group with the highest flow ratio in each phase is not necessarily the critical lane group for that phase. The remainder of this chapter will focus only on nonoverlapping phases. The reader is referred to the *Highway Capacity Manual* [Transportation Research Board 2000] for details on determining critical lane groups for overlapping phases.

In addition, the sum of the flow ratios for the critical lane groups can be used to calculate a suitable cycle length, which will be discussed in the next section. This is given by

$$Y_c = \sum_{i=1}^{n}\left(\frac{v}{s}\right)_{ci} \tag{7.18}$$

where

Y_c = sum of flow ratios for critical lane groups,
$(v/s)_{ci}$ = flow ratio for critical lane group i, and
n = number of critical lane groups.

The total lost time for the cycle will also be used in the calculation of cycle length. In determining the total lost time for the cycle, the general rule is to apply the lost time for a critical lane group when its movements are initiated (the start of its green interval). The total cycle lost time is given as

$$L = \sum_{i=1}^{n}(t_L)_{ci} \tag{7.19}$$

where

L = total lost time for cycle in seconds,
$(t_L)_{ci}$ = total lost time for critical lane group i in seconds, and
n = number of critical lane groups.

EXAMPLE 7.8

Calculate the sum of the flow ratios for the critical lane groups for the three-phase timing plan determined in Example 7.6 given the saturation flow rates in Table 7.1.

SOLUTION

Note that the saturation flow rates are relatively low for the SB and NB left (L) turns because they are permitted only, and the opposing through and right-turn (T/R) vehicles limit the number of usable gaps for these vehicles. The saturation flow rates for the WB and EB through and right turn (T/R) movements account for both through lanes.

The flow ratios will now be calculated, with the critical lane group for each phase indicated with a check mark in Table 7.2.

Table 7.1 Saturation Flow Rates for Three-Phase Design at Intersection of Maple Street and Vine Street

Phase 1	Phase 2	Phase 3
EB L: 1750 veh/h	EB T/R: 3400 veh/h	SB L: 450 veh/h NB L: 475 veh/h
WB L: 1750 veh/h	WB T/R: 3400 veh/h	SB T/R: 1800 veh/h NB T/R: 1800 veh/h

Table 7.2 Flow Ratios and Critical Lane Groups for Three-Phase Design at Intersection of Maple Street and Vine Street

Phase 1	Phase 2	Phase 3
EB L: $\frac{300}{1750} = 0.171$ √	EB T/R: $\frac{1100}{3400} = 0.324$	SB L: $\frac{70}{450} = 0.156$
		NB L: $\frac{90}{475} = 0.189$
WB L: $\frac{250}{1750} = 0.143$	WB T/R: $\frac{1150}{3400} = 0.338$ √	SB T/R: $\frac{370}{1800} = 0.206$
		NB T/R: $\frac{390}{1800} = 0.217$ √

As indicated in the table, the critical lane group for phases 1, 2, and 3, respectively, are the EB left turn, the WB through and right turn, and the NB through and right turn.

The sum of the flow ratios for the critical lane groups for this phasing plan will be needed for the next section. Since this phasing plan does not include any overlapping phases, this value is simply the sum of the highest lane group v/s ratios for the three phases, as follows:

$$Y_c = \sum_{i=1}^{n} \left(\frac{v}{s}\right)_{ci}$$

$$= 0.171 + 0.338 + 0.217 = \underline{\underline{0.726}}$$

Assuming 2 seconds of start-up lost time and 2 seconds of clearance lost time (1 second of yellow time plus 1 second of all-red time), for each critical lane group, gives a lost time of 4 s/phase. The total lost time for the cycle is then 12 seconds (3 phases × 4 s/phase).

EXAMPLE 7.9

Suppose it is necessary to run the NB and SB movements in a split-phase configuration (with phase 3 for SB movements and a new phase 4 for NB movements). Calculate the sum of the flow ratios for the critical lane groups and total cycle lost time for this situation, assuming the EB and WB movement phasing remains the same.

Table 7.3 summarizes the calculation of the flow ratios and the identification of the critical lane groups.

SOLUTION

The sum of the flow ratios for the critical lane groups for this phasing plan is

$$\sum_{i=1}^{n} \left(\frac{v}{s}\right)_{ci} = 0.171 + 0.338 + 0.206 + 0.217 = \underline{\underline{0.932}}$$

The total lost time for the cycle is $\underline{\underline{16 \text{ seconds}}}$ (4 phases × 4 s/phase).

Table 7.3 Flow Ratios and Critical Lane Groups for Four-Phase Design (Split Phase for N-S Movements) at Intersection of Maple Street and Vine Street

Phase 1	Phase 2	Phase 3	Phase 4
EB L: $\frac{300}{1750} = 0.171$ √	EB T/R: $\frac{1100}{3400} = 0.324$	SB L: $\frac{70}{1750} = 0.040$	NB L: $\frac{90}{1750} = 0.051$
WB L: $\frac{250}{1750} = 0.143$	WB T/R: $\frac{1150}{3400} = 0.338$ √	SB T/R: $\frac{370}{1800} = 0.206$ √	NB T/R: $\frac{390}{1800} = 0.217$ √

7.5.5 Calculate Cycle Length

The cycle length is simply the summation of the individual phase lengths. In practice, cycle lengths are generally kept as short as possible, typically between 40 and 60 seconds. However, complex intersections with five or more phases can have cycle lengths of 120 seconds or more. The minimum cycle length necessary for the lane group volumes and phasing plan of an intersection is given by

$$C_{min} = \frac{L \times X_c}{X_c - \sum_{i=1}^{n} \left(\frac{v}{s}\right)_{ci}} \tag{7.20}$$

where

C_{min} = minimum necessary cycle length in seconds (typically rounded up to the nearest 5-second increment in practice),
L = total lost time for cycle in seconds,
X_c = critical v/c ratio for the intersection,
$(v/s)_{ci}$ = flow ratio for critical lane group i, and
n = number of critical lane groups.

In this equation, the total lost time for the cycle and the sum of the flow ratios for the critical lane groups are predetermined. However, a critical intersection volume/capacity ratio, X_c, must be chosen for the desired degree of utilization. In other words, if it is desired for the intersection to operate at its full capacity, a value of 1.0 is used for X_c. A value of 1.0 is not generally recommended, however, due to the randomness of vehicle arrivals, which can result in occasional cycle failures. Note that this equation gives the minimum cycle length necessary for the intersection to operate at a specified degree of capacity utilization. This cycle length does not necessarily minimize the average vehicle delay experienced by motorists at the intersection.

A practical equation for the calculation of the cycle length that seeks to minimize vehicle delay was developed by Webster [1958]. Webster's optimum cycle length formula is

$$C_{opt} = \frac{1.5 \times L + 5}{1.0 - \sum_{i=1}^{n} \left(\frac{v}{s}\right)_{ci}} \tag{7.21}$$

where

C_{opt} = cycle length to minimize delay in seconds, and
Other terms are as defined previously.

The cycle length determined from this calculation is only approximate. Webster noted that values between $0.75C_{opt}$ and $1.5C_{opt}$ will likely give similar values of delay. Calculating an accurate optimal cycle length (and phase length) can be a very computationally intensive exercise for all but the most simple signalized intersections, especially if coordination between multiple signals is involved.

It should be noted that regardless of the minimum or optimal cycle length calculated, practical maximum cycle lengths must generally be observed. Public acceptance or tolerance of large cycle lengths will vary by location (urban vs. rural), but as a rule, cycle lengths in excess of 3 minutes (180 seconds) should be used only in exceptional circumstances.

EXAMPLE 7.10

Calculate the minimum and optimal cycle lengths for the intersection of Maple and Vine streets, using the information provided in the preceding examples, for both the three-phase and four-phase design.

SOLUTION

For the three-phase design (Example 7.8), the sum of the flow ratios for the critical lane groups and the total cycle lost time were determined to be 0.726 and 12 seconds, respectively. For the minimum cycle length, a somewhat conservative value of 0.9 will be used for the critical intersection v/c ratio to minimize the potential of cycle failures due to occasionally high arrival volumes. Using these values in Eq. 7.20 gives

$$C_{min} = \frac{12 \times 0.9}{0.9 - 0.726} = 62.1 \rightarrow \underline{\underline{65 \text{ s}}} \text{ (rounding up to nearest 5 seconds)}$$

Using Eq. 7.21 for the optimal cycle length gives

$$C_{opt} = \frac{1.5 \times 12 + 5}{1.0 - 0.726} = 83.9 \rightarrow \underline{\underline{85 \text{ s}}} \text{ (rounding up to nearest 5 seconds)}$$

For the four-phase design (Example 7.9), the sum of the critical flow ratios and the total cycle lost time were determined to be 0.932 and 16 seconds, respectively. The first issue with this design is that a higher X_c will need to be used because the sum of flow ratios for critical lane groups is higher than the 0.90 used for the three-phase design (otherwise the denominator of Eq. 7.20 will be negative). To minimize the cycle length, the maximum value of 1.0 will be used for X_c in Eq. 7.20, as follows:

$$C_{min} = \frac{16 \times 1.0}{1.0 - 0.932} = \underline{\underline{235.3 \text{ s}}}$$

The second issue is that despite the use of an X_c value of 1.0 (the intersection operating at capacity) to minimize the cycle length, an unreasonably high cycle length is still required for this design. Thus, this design is not nearly as desirable as the three-phase design.

Generally a split-phase design is recommended only under one or more of the following conditions:

- The left turns are the dominant movement.
- The left turns share a lane with the through movement.

- There is a large difference in the total approach volumes.
- There are unusual opposing approach geometrics.

It should also be noted that serving pedestrians in an efficient manner on split-phase approaches can be difficult.

7.5.6 Allocate Green Time

After a cycle length has been calculated, the next step in the traffic signal timing process is to determine how much green time should be allocated to each phase. The cycle length is the sum of all effective green times plus the total lost time. Thus, after subtracting the total lost time from the cycle length, the remaining time can be distributed as green time among the phases of the cycle.

There are several strategies for allocating the green time to the various phases. One of the most popular and simplest is to distribute the green time such that the v/c ratios are equalized for the critical lane groups, as by the following equation:

$$g_i = \left(\frac{v}{s}\right)_{ci}\left(\frac{C}{X_i}\right) \tag{7.22}$$

where

g_i = effective green time for phase i,
$(v/s)_{ci}$ = flow ratio for critical lane group i,
C = cycle length in seconds, and
X_i = v/c ratio for lane group i.

EXAMPLE 7.11

Determine the green-time allocations for the 65-second cycle length found in Example 7.10, using the method of v/c ratio equalization.

SOLUTION

Because the calculated cycle length was rounded up a few seconds, the critical intersection v/c ratio for this rounded cycle length will be calculated for use in the green-time allocation calculations. Equation 7.20 can be rearranged to solve for X_c as follows:

$$X_c = \frac{\sum_{i=1}^{n}\left(\frac{v}{s}\right)_{ci} \times C}{C - L}$$

Using this equation with

$(v/s)_{ci}$ = 0.726 (Example 7.8)
C = 65 s (Example 7.10)
L = 12 s (Example 7.8)

gives

$$X_c = \frac{0.726 \times 65}{65 - 12} = 0.890$$

Therefore, the cycle length of 65 seconds and X_c of 0.890 are used to calculate the effective green times for the three phases, as follows:

$$g_1 = \left(\frac{v}{s}\right)_{c1}\left(\frac{C}{X_1}\right)$$

$$= 0.171 \times \frac{65}{0.890} = \underline{\underline{12.5\text{ s}}} \quad \text{(EB and WB left-turn movements)}$$

$$g_2 = \left(\frac{v}{s}\right)_{c2}\left(\frac{C}{X_2}\right)$$

$$= 0.338 \times \frac{65}{0.890} = \underline{\underline{24.7\text{ s}}} \quad \text{(EB and WB through and right-turn movements)}$$

$$g_3 = \left(\frac{v}{s}\right)_{c3}\left(\frac{C}{X_3}\right)$$

$$= 0.217 \times \frac{65}{0.890} = \underline{\underline{15.8\text{ s}}} \quad \text{(NB and SB left, through, and right-turn movements)}$$

The cycle length is checked by summing these effective green times and the lost time, giving

$$C = g_1 + g_2 + g_3 + L$$
$$= 12.5 + 24.7 + 15.8 + 12 = 65.0$$

Therefore, all calculations are correct.

7.5.7 Calculate Change and Clearance Intervals

Recall that the change interval corresponds to the yellow time and the clearance interval corresponds to the all-red time. If an all-red interval does not exist, then the yellow time is considered as both the change and clearance intervals. The change interval alerts drivers that the green interval is about to end and that they should come to stop before entering the intersection, or continue through the intersection if they are too close to come to a safe stop. The clearance interval allows for those vehicles that might have entered the intersection at the end of the yellow to clear the intersection before conflicting traffic movements are given a green signal indication. In the past, the yellow indication was intended to also allow for clearance time. Today, however, there is routine red-indication abuse and frequent running of red indications after the yellow time. As a result, the all-red indication is often implemented.

Typically, the yellow time is in the range of 3 to 5 seconds. Warning times that are shorter than 3 seconds and longer than 5 seconds are not practical because long warning times encourage motorists to continue to enter the intersection whereas short times can place the driver in a dilemma zone. A dilemma zone is created for the driver if a safe stop before the intersection cannot be accomplished, and continuing through the intersection at a constant speed (without accelerating) will result in the vehicle entering the intersection during a red indication. If a dilemma zone exists, drivers always make the wrong decision, whether they decide to stop or to continue through the intersection. Figure 7.13 illustrates the dilemma zone. Referring to this figure, suppose a vehicle traveling at a constant speed requires distance x_s to stop. If the vehicle is closer to the intersection than distance d_d, then it can enter before the all-red indication. If the vehicle is in the shaded area ($x_s - d_d$ from the intersection) when the yellow light is displayed, the driver is in the dilemma zone and can neither stop in time nor continue through the intersection at a constant speed without passing through a red indication.

Formulas and policies for calculating yellow (Y) and all-red (AR) times vary by agency, but one set of commonly accepted formulas is provided in the *Traffic Engineering Handbook* [ITE 1999] and are as follows:

$$Y = t_r + \frac{V}{2a + 2gG} \tag{7.23}$$

where

Y = yellow time (usually rounded up to the nearest 0.5 second),
t_r = driver perception/reaction time, usually taken as 1.0 second,
V = speed of approaching traffic in ft/s (m/s),
a = deceleration rate for the vehicle, usually taken as 10.0 ft/s^2 (3.05 m/s^2),
g = acceleration due to gravity [32.2 ft/s^2 (9.807 m/s^2)], and
G = percent grade divided by 100.

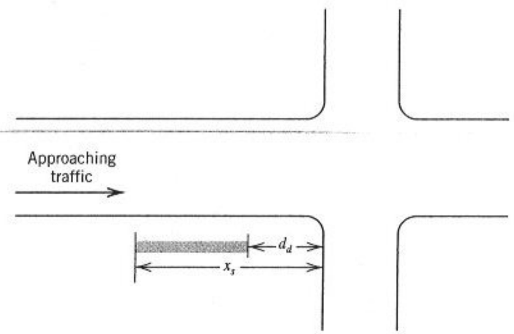

Figure 7.13 The dilemma zone for traffic approaching a signalized intersection.

and

$$AR = \frac{w+l}{V} \tag{7.24}$$

where

AR = all-red time (usually rounded up to the nearest 0.5 second),
w = width of the cross street in ft (m),
l = length of the vehicle, usually taken as 20 ft (6 m), and
V = speed of approaching traffic in ft/s (m/s).

To avoid a dilemma zone and the possibility of a vehicle being in the intersection when a conflicting movement receives a green-signal indication, the total of the change and clearance intervals (yellow plus all-red times) should always be equal to or greater than the sum of Eqs. 7.23 and 7.24.

EXAMPLE 7.12

Determine the yellow and all-red times for vehicles traveling on Vine and Maple streets as shown in Fig 7.9.

SOLUTION

For the Vine Street phasing (applying Eqs. 7.23 and 7.24),

$$Y = 1.0 + \frac{(35 \times 5280/3600)}{2(10)}$$

$$= 3.6 \quad \rightarrow \quad \underline{\underline{4.0 \text{ s}}} \text{ (rounding up to the nearest 0.5 s)}$$

$$AR = \frac{60+20}{35 \times 5280/3600}$$

$$= 1.6 \quad \rightarrow \quad \underline{\underline{2.0 \text{ s}}} \text{ (rounding up to the nearest 0.5 s)}$$

For the Maple Street phasing (applying Eqs. 7.23 and 7.24),

$$Y = 1.0 + \frac{(40 \times 5280/3600)}{2(10)}$$

$$= 3.9 \quad \rightarrow \quad \underline{\underline{4.0 \text{ s}}} \text{ (rounding up to the nearest 0.5 s)}$$

$$AR = \frac{36+20}{40 \times 5280/3600}$$

$$= \underline{\underline{1.0 \text{ s}}}$$

Note that separate calculations are usually required for exclusive left-turn phases, as vehicle approach speeds are often lower than for through vehicles and intersection crossing distances may be longer (due to the width of the opposing direction and the circular travel path).

7.5.8 Check Pedestrian Crossing Time

In urban areas and other locations where pedestrians are present, the signal-timing plan should be checked for its ability to provide adequate pedestrian crossing time. At locations where streets are wide and green times are short, it is possible that pedestrians can be caught in the middle of the intersection when the phase changes. To avoid this problem, the minimum green time required for pedestrian crossing should be checked against the apportioned green time for the phase. If there is not enough green time for a pedestrian to safely cross the street, the apportioned green time should be increased to meet the pedestrian needs. If pedestrian pushbuttons are provided at an intersection (for actuated control), the green time can be increased to meet pedestrian crossing needs only when the pushbuttons are activated.

The minimum pedestrian green time is given by

$$G_p = 3.2 + \frac{L}{S_p} + (0.27N_{ped}) \quad \text{for } W_E \leq 10 \text{ ft (3.05 m)} \tag{7.25}$$

$$G_p = 3.2 + \frac{L}{S_p} + \left(2.7\frac{N_{ped}}{W_E}\right) \quad \text{for } W_E > 10 \text{ ft (3.05 m)} \tag{7.26}$$

where

G_p = minimum pedestrian green time in seconds,
3.2 = pedestrian start-up time in seconds,
L = crosswalk length in ft (m),
S_p = walking speed of pedestrians, usually taken as 4.0 ft/s (1.2 m/s),
N_{ped} = number of pedestrians crossing during an interval, and
W_E = effective crosswalk width in ft (m).

The generally recommended walking speed of 4.0 ft/s (1.2 m/s) represents a slower-than-average speed. However, at intersections where a significant number of slower pedestrians (elderly, vision impaired, etc.) are served, the use of a slower walking speed may be warranted.

EXAMPLE 7.13

Determine the minimum amount of pedestrian green time required for the intersection of Vine and Maple streets. Assume a maximum of 15 pedestrians crossing either street during any one phase and a crosswalk width of 8 ft (2.44 m).

SOLUTION

A pedestrian who crosses Maple Street will cross while Vine Street has a green interval. The minimum pedestrian green time needed on Vine Street is (using Eq. 7.25, as the effective crosswalk width is less than or equal to 10 ft)

$$G_p = 3.2 + \frac{36}{4.0} + (0.27 \times 15) = \underline{\underline{16.25 \text{ s}}}$$

In Example 7.11, Vine Street was assigned 15.8 seconds of effective green time [13.8 seconds of displayed green time (from Eq. 7.3)]. This amount of time is insufficient for pedestrians crossing Maple Street. Therefore, the green time for this phase will have to be increased to accommodate crossing pedestrians, and the overall signal timing plan adjusted accordingly (although we will continue to use the previously computed green time in subsequent examples). The minimum pedestrian green time needed on Maple Street (for the through/right-turn phase, when pedestrian movement would be permitted) is

$$G_p = 3.2 + \frac{60}{4.0} + (0.27 \times 15) = \underline{\underline{22.25 \text{ s}}}$$

In Example 7.11, Maple Street was assigned 24.7 seconds of effective green time (23.7 seconds of displayed green time) for this phase, so this green time is adequate for pedestrians crossing Vine Street.

7.6 LEVEL-OF-SERVICE DETERMINATION

Before the implementation of any developed signal phasing and timing plan, the level of service should be determined to assess whether the intersection will operate at an acceptable level under this plan. As previously mentioned, the service measure (the performance measure by which level of service is assessed) for signalized intersections is delay. In previous examples we calculated the delay for a specific lane group, but now we must do this for all lane groups and then aggregate the delay values to arrive at an overall intersection delay measure and corresponding level of service.

The first step is to aggregate the delays of all lane groups for an approach, and then repeat the procedure for each approach of the intersection. This will result in approach-specific delays and levels of service. The aggregated lane group delay for each approach is given by

$$d_A = \frac{\sum_i d_i v_i}{\sum_i v_i} \tag{7.27}$$

where

d_A = average delay per vehicle for approach A in seconds,
d_i = average delay per vehicle for lane group i (on approach A) in seconds, and
v_i = analysis flow rate for lane group i in veh/h.

Once all the approach delays have been calculated, they can be aggregated to arrive at the overall intersection delay. The aggregated approach delay for the intersection is given by

$$d_I = \frac{\sum_A d_A v_A}{\sum_A v_A} \tag{7.28}$$

where

d_I = average delay per vehicle for the intersection in seconds,
d_A = average delay per vehicle for approach A in seconds, and
v_A = analysis flow rate for approach A in veh/h.

The delay level-of-service criteria for signalized intersections are specified in the *Highway Capacity Manual* [Transportation Research Board 2000] and are given in Table 7.4. These delay criteria can be used to determine the level of service for a lane group, an approach, and the intersection.

Table 7.4 Level-of-Service Criteria for Signalized Intersections

LOS	Control delay per vehicle (s/veh)
A	≤ 10
B	> 10–20
C	> 20–35
D	> 35–55
E	> 55–80
F	> 80

Source: Transportation Research Board. *Highway Capacity Manual.* Washington, DC: National Research Council, 2000.

EXAMPLE 7.14

Determine the level of service for the eastbound approach of Maple Street assuming no initial queue at the start of the analysis period.

SOLUTION

Lane groups for this intersection were established in Example 7.7. Two lane groups were established for the eastbound (EB) approach, one for the left-turn movement and the other for the combined through and right-turn movements. The delay will be calculated for the left-turn lane group first, followed by the through/right-turn lane group.

For the left-turn lane group, the uniform delay is computed using Eq. 7.15 with

$$C = 65 \text{ s}$$
$$g = 12.5 \text{ s}$$
$$X = \frac{v}{c} = \frac{v}{s \times g/C} = \frac{300}{1750 \times 12.5/65} = \frac{300}{337} = 0.891$$

giving

$$d_1 = \frac{0.5(65)\left(1 - \frac{12.5}{65}\right)^2}{1 - \left[0.891 \times \frac{12.5}{65}\right]} = 25.6 \text{ s}$$

Random delay is computed using Eq. 7.16 with

$T = 0.25$ (15 min)
$X = 0.891$ (from above)
$k = 0.5$ (pretimed control)
$I = 1.0$ (isolated mode)
$c = 337$ veh/h (from above)

giving

$$d_2 = 900(0.25)\left[(0.891 - 1) + \sqrt{(0.891 - 1)^2 + \frac{8(0.5)(1.0)0.891}{(337)0.25}}\right]$$
$$= 27.8 \text{ s}$$

Now, the total signal delay is computed using Eq. 7.14 with

$d_1 = 25.6$ s
$PF = 1.0$ (progression neutral)
$d_2 = 27.8$ s
$d_3 = 0$ s (given)

$$d = 25.6 \times 1.0 + 27.8 + 0 = 53.4 \text{ s}$$

From Table 7.4, this lane group delay corresponds to a level of service of D.

For the through/right-turn lane group, the uniform delay is computed using Eq. 7.15 with

$$C = 65 \text{ s}$$
$$g = 24.7 \text{ s}$$
$$X = \frac{v}{c} = \frac{v}{s \times g/C} = \frac{1100}{3400 \times 24.7/65} = \frac{1100}{1292} = 0.851$$

giving

$$d_1 = \frac{0.5(65)\left(1 - \frac{24.7}{65}\right)^2}{1 - \left[0.851 \times \frac{24.7}{65}\right]} = 18.5 \text{ s}$$

Random delay is computed using Eq. 7.16 with

$T = 0.25$ (15 min)
$X = 0.851$ (from above)
$k = 0.5$ (pretimed control)
$I = 1.0$ (isolated mode)
$c = 1292$ veh/h (from above)

giving

$$d_2 = 900(0.25)\left[(0.851 - 1) + \sqrt{(0.851 - 1)^2 + \frac{8(0.5)(1.0)0.851}{(1292)0.25}}\right]$$
$$= 7.2 \text{ s}$$

Now, the average signal delay for this lane group is computed using Eq. 7.14 with

$d_1 = 18.5$ s
$PF = 1.0$ (progression neutral)
$d_2 = 7.2$ s
$d_3 = 0$ s (given)

$$d = 18.5 \times 1.0 + 7.2 + 0 = 25.7 \text{ s}$$

From Table 7.4, this lane group delay corresponds to a level of service of C.

Now, to compute the volume-weighted aggregate delay for the approach, we use Eq. 7.27:

$$d_A = \frac{\sum d_i v_i}{\sum v_i}$$

with

d_{LT} = delay for left-turn lane group
$d_{T/R}$ = delay for through/right-turn lane group
v_{LT} = analysis flow rate for left-turn lane group
$v_{T/R}$ = analysis flow rate for through/right-turn lane group

giving

$$d_{EB} = \frac{v_{LT} \times d_{LT} + v_{T/R} \times d_{T/R}}{v_{LT} + v_{T/R}}$$

$$= \frac{300 \times 53.4 + 1100 \times 25.7}{300 + 1100}$$

$$= \frac{44{,}290}{1400}$$

$$= \underline{\underline{31.6 \text{ s}}}$$

From Table 7.4, this approach delay corresponds to a level of service of C.

EXAMPLE 7.15

Determine the level of service for the intersection of Maple and Vine streets.

SOLUTION

The delay for each of the other three approaches (WB, NB, SB) can be determined by the exact same process used in Example 7.14 for the EB approach. Due to the length of the calculations involved for the remaining three approaches, the results are summarized in Table 7.5.

In this table, note that the v/c ratios for the critical lane groups match (rounding differences aside) the calculated critical intersection v/c ratio, X_c, as they should, because green time was allocated based on the strategy of equalizing v/c ratios for the critical lane groups in each phase (using Eq. 7.22).

The overall intersection delay calculation will be shown for the sake of clarity. Using Eq. 7.28, the intersection delay is given by

$$d_I = \frac{31.6 \times 1400 + 30.2 \times 1400 + 49.7 \times 480 + 42.3 \times 440}{1400 + 1400 + 480 + 440}$$

$$= \frac{128{,}988}{3{,}720}$$

$$= \underline{\underline{34.7 \text{ s}}}$$

It is worth pointing out that although all but two lane groups (EB T/R and WB T/R) have a level of service of D or higher, the much higher volumes for those two approaches relative to the others keeps the level of service at C (albeit barely) due to the volume weighting in the delay aggregation.

Table 7.5 Summary of Delay and Level-of-Service Calculations for the Intersection of Maple and Vine Streets

Approach	EB		WB		NB		SB	
Lane group	LT	T/R	LT	T/R	LT	T/R	LT	T/R
Analysis flow rate (v)	300	1100	250	1150	90	390	70	370
Saturation flow rate (s)	1750	3400	1750	3400	475	1800	450	1800
Flow ratio (v/s)	0.171	0.324	0.143	0.338	0.189	0.217	0.156	0.206
Critical lane group (√)	√			√		√		
Y_c	0.726							
Lost time/phase	4							
Total lost time	12							
Cycle length (C_{min})	65							
X_c	0.891							
Eff. green time (g)	12.5	24.7	12.5	24.7	15.8	15.8	15.8	15.8
g/C	0.192	0.380	0.192	0.380	0.243	0.243	0.243	0.243
Lane group capacity (c)	337	1292	337	1292	115	438	109	438
v/c (X)	0.891	0.851	0.743	0.890	0.779	0.891	0.640	0.846
d_1	25.6	18.5	24.7	18.9	23.0	23.8	22.1	23.4
PF	1.0							
k	0.5							
I	1.0							
T	0.25							
d_2	27.8	7.2	13.8	9.5	39.4	23.0	25.3	17.9
Lane group delay (d)	53.4	25.7	38.5	28.3	62.4	46.7	47.3	41.4
Lane group LOS	D	C	D	C	E	D	D	D
Approach delay	31.6		30.2		49.7		42.3	
Approach LOS	C		C		D		D	
Intersection delay	34.7							
Intersection LOS	C							

Tables 18.1 and 18.2, respectively.

TRAFFIC ENG. RPM

Table 18.1: Through Vehicle Equivalents for Left-Turning Vehicles, E_{LT}

Opposing Flow V_o (veh/h)	Number of Opposing Lanes, N_o: 1	2	3
0	1.1	1.1	1.1
200	2.5	2.0	1.8
400	5.0	3.0	2.5
600	10.0*	5.0	4.0
800	13.0*	8.0	6.0
1,000	15.0*	13.0*	10.0*
≥1,200	15.0*	15.0*	15.0*

E_{LT} for all *protected* left turns = 1.05

*indicates that the LT capacity is only available through "sneakers."

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Once ap been selected, converted to Subsequently, each approach