CHAPTER 18
SIGNALIZED INTERSECTIONS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Chapter 18, Signalized Intersections, describes a methodology for evaluating the capacity and quality of service provided to road users traveling through a signalized intersection. However, the methodology is much more than just a tool for evaluating capacity and quality of service. It includes an array of performance measures that describe intersection operation for multiple travel modes. These measures serve as clues for identifying the source of problems and provide insight into the development of effective improvement strategies. The analyst using this methodology is encouraged to consider the full range of measures.

OVERVIEW OF THE METHODOLOGY

This chapter's methodology applies to three- and four-leg intersections of two streets or highways where the signalization operates in isolation from nearby intersections.

The influence of an upstream signalized intersection on the subject intersection's operation is addressed by input variables that describe platoon structure and the uniformity of arrivals on a cyclic basis. Chapter 17, Urban Street Segments, describes a methodology for evaluating an intersection that is part of a coordinated signal system.

Analysis Boundaries

The intersection analysis boundaries are not defined at a fixed distance for all intersections. Rather, they are dynamic and extend backward from the intersection a sufficient distance to include the operational influence area on each intersection leg. The size of this area is leg-specific and includes the most distant extent of any intersection-related queue expected to occur during the study period. For these reasons, the analysis boundaries should be established for each intersection according to conditions during the analysis period. The influence area should extend at least 250 ft back from the stop line on each intersection leg.

Analysis Level

Analysis level describes the level of detail used when the methodology is applied. Three levels are recognized:

- Operational,
- Design, and
- Planning and preliminary engineering.

The operational analysis is the most detailed application and requires the most information about traffic, geometric, and signalization conditions. The design analysis also requires detailed information about traffic conditions and the desired level of service (LOS) as well as information about geometric or signalization conditions. The design analysis then seeks to determine reasonable values for the conditions not provided. The planning and preliminary engineering analysis requires only the most fundamental types of information from the analyst. Default values are then used as substitutes for other input data. Analysis level is discussed in more detail in the applications section of this chapter.

Study Period and Analysis Period

The study period is the time interval represented by the performance evaluation. It consists of one or more consecutive analysis periods. An analysis period is the time interval evaluated by a single application of the methodology.

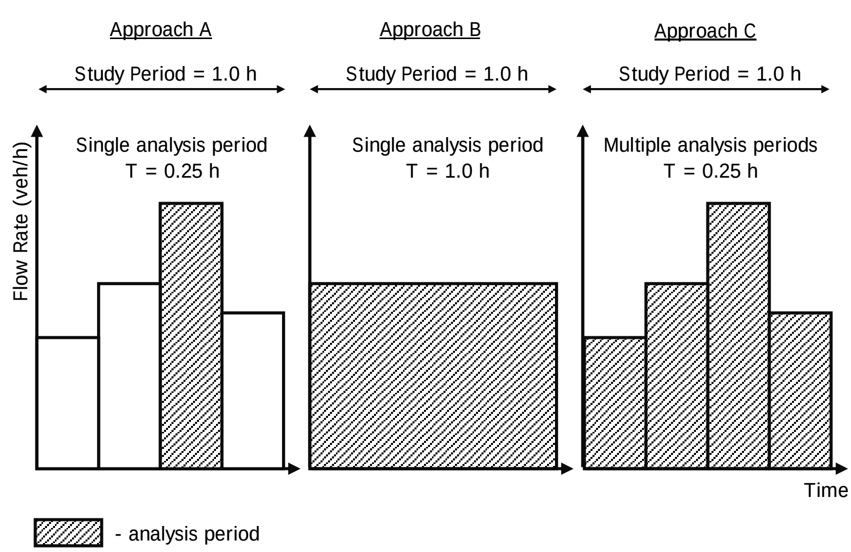
The methodology is based on the assumption that traffic conditions are steady during the analysis period (i.e., systematic change over time is negligible). For this reason, the analysis period ranges from 0.25 to 1 h. The longer durations are sometimes used for planning analyses. In general, the analyst should use caution with analysis periods that exceed 1 h because traffic conditions typically are not steady for long time periods and because the adverse impact of short peaks in traffic demand may not be detected in the evaluation.

If an analysis period of interest has a demand volume that exceeds capacity, then the study period should include an initial analysis period with no initial queue and a final analysis period with no residual queue. This approach provides a more accurate estimate of the delay associated with the congestion.

If evaluation of multiple analysis periods is determined to be important, then the performance estimates for each period should be reported separately. In this situation, reporting an average performance for the study period is not encouraged because it may obscure extreme values and suggest acceptable operation when some analysis periods have unacceptable operation.

Exhibit 18-1 demonstrates three alternative approaches an analyst might use for a given evaluation. Other alternatives exist, and the study period can exceed 1 h. Approach A has traditionally been used and, unless otherwise justified, is the one recommended for use.

Exhibit 18-1
Three Alternative Study Approaches



Approach A is based on evaluation of the peak 15-min period during the study period. The analysis period T is 0.25 h. The equivalent hourly flow rate in vehicles per hour (veh/h) used for the analysis is based on either a peak 15-min traffic count multiplied by four or a 1-h demand volume divided by the peak hour factor. The former option is preferred when traffic counts are available. Additional discussion on use of the peak hour factor is provided in the required input data subsection.

Approach B is based on evaluation of one 1-h analysis period that is coincident with the study period. The analysis period *T* is 1.0 h. The flow rate used is equivalent to the 1-h demand volume (i.e., the peak hour factor is not used). This approach implicitly assumes that the arrival rate of vehicles is constant throughout the period of study. Therefore, the effects of peaking within the hour may not be identified and the analyst risks underestimating the delay actually incurred.

Approach C uses a 1-h study period and divides it into four 0.25-h analysis periods. This approach accounts for systematic flow rate variation among analysis periods. It also accounts for queues that carry over to the next analysis period and produces a more accurate representation of delay.

Performance Measures

An intersection's performance is described by the use of one or more quantitative measures that characterize some aspect of the service provided to a specific road user group. Performance measures cited in this chapter include automobile volume-to-capacity ratio, automobile delay, queue storage ratio, pedestrian delay, pedestrian circulation area, pedestrian perception score, bicycle delay, and bicycle perception score.

LOS is also considered a performance measure. It is computed for the automobile, pedestrian, and bicycle travel modes. It is useful for describing intersection performance to elected officials, policy makers, administrators, and the public. LOS is based on one or more of the performance measures listed in the preceding paragraph.

Travel Modes

This chapter describes three methodologies that can be used to evaluate intersection performance from the perspective of motorists, pedestrians, and bicyclists. They are referred to as the automobile methodology, the pedestrian methodology, and the bicycle methodology.

The automobile methodology has evolved and reflects the findings from a large body of research. It was originally based, in part, on the results of a National Cooperative Highway Research Program (NCHRP) study (*1*, *2*) that formalized the critical movement analysis procedure and the automobile delay estimation procedure. The critical movement analysis procedure was developed in the United States (*3*, *4*), Australia (*5*), Great Britain (*6*), and Sweden (*7*). The automobile delay estimation procedure was developed in Great Britain (*8*), Australia (*9*), and the United States (*10*). Updates to the original methodology were developed in a series of research projects (*11–24*). The procedures for evaluating pedestrian and bicyclist perception of LOS are documented in an NCHRP report (*25*). The procedures for evaluating pedestrian delay, pedestrian circulation area, and bicyclist delay are documented in two Federal Highway Administration reports (*26, 27*).

The phrase *automobile mode*, as used in this chapter, refers to travel by all motorized vehicles that can legally operate on the street, with the exception of local transit vehicles that stop to pick up passengers at the intersection. Unless explicitly stated otherwise, the word *vehicles* refers to motorized vehicles and includes a mixed stream of automobiles, motorcycles, trucks, and buses.

Lane Groups and Movement Groups

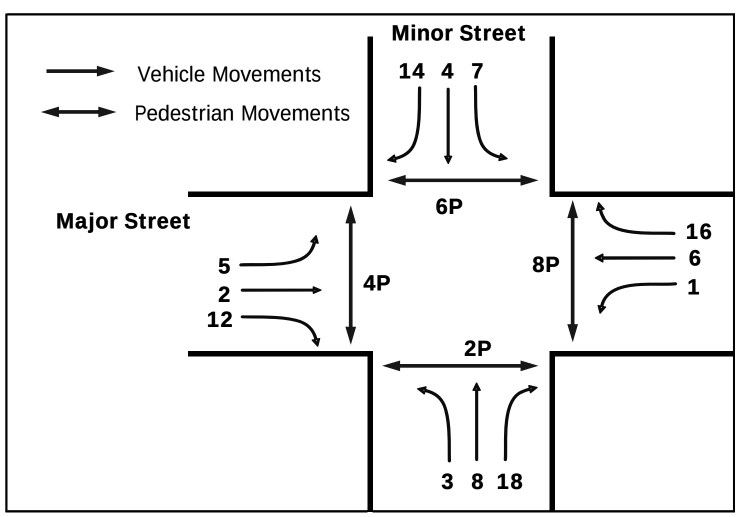
The automobile methodology is designed to evaluate the performance of designated lanes, groups of lanes, an intersection approach, and the entire intersection. A lane or group of lanes designated for separate analysis is referred to as a *lane group*. In general, a separate lane group is established for (*a*) each lane (or combination of adjacent lanes) that exclusively serves one movement and (*b*) each lane shared by two or more movements. Guidelines for establishing lane groups are described in Section 2, Methodology.

The concept of *movement groups* is also established to facilitate data entry. A separate movement group is established for (*a*) each turn movement with one or more exclusive turn lanes and (*b*) the through movement (inclusive of any turn movements that share a lane).

Movement and Phase Numbering

Exhibit 18-2 illustrates the vehicle and pedestrian traffic movements at a four-leg intersection. Three vehicular traffic movements and one pedestrian traffic movement are shown for each intersection approach. To facilitate the discussion in this chapter, each movement is assigned a unique number or a number and letter combination. The letter P denotes a pedestrian movement.

Exhibit 18-2
Intersection Traffic Movements and Numbering Scheme

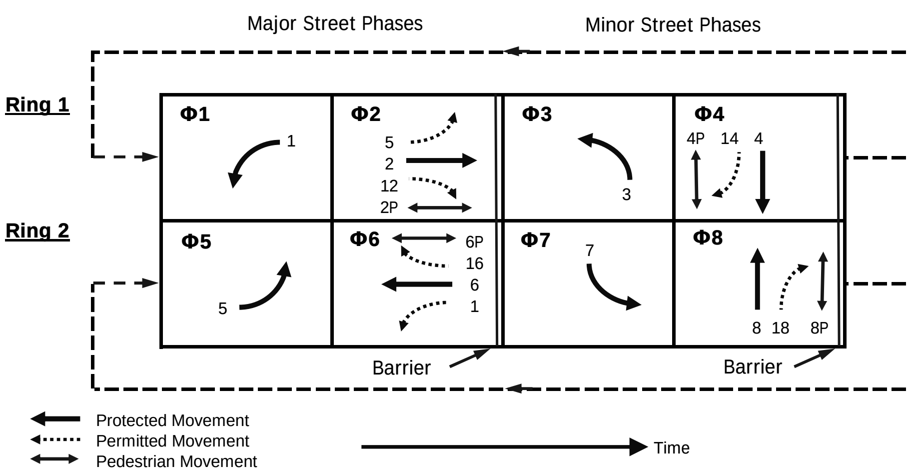


Modern actuated controllers implement signal phasing by using a dual-ring structure that allows for the concurrent presentation of a green indication to two phases. Each phase serves one or more movements that do not conflict with each other. The commonly used eight-phase dual-ring structure is shown in Exhibit

18-3. The symbol Φ shown in this exhibit represents the word "phase," and the number following the symbol represents the phase number.

Exhibit 18-3 shows one way that traffic movements can be assigned to each of the eight phases. These assignments are illustrative, but they are not uncommon. Each left-turn movement is assigned to an exclusive phase. During this phase, the left-turn movement is "protected" so that it receives a green arrow indication. Each through, right-turn, and pedestrian movement combination is also assigned to an exclusive phase. The dashed arrows indicate turn movements that are served in a "permitted" manner so that the turn can be completed only after yielding the right-of-way to conflicting movements. Additional information about traffic signal controller operation is provided in Chapter 31, Signalized Intersections: Supplemental.

Exhibit 18-3
Dual-Ring Structure with Illustrative Movement Assignments



LOS CRITERIA

This subsection describes the LOS criteria for the automobile, pedestrian, and bicycle modes. The criteria for the automobile mode are different from those for the nonautomobile modes. Specifically, the automobile-mode criteria are based on performance measures that are field measurable and perceivable by travelers. The criteria for the nonautomobile modes are based on scores reported by travelers indicating their perception of service quality.

Automobile Mode

All uses of the word "volume" or the phrase "volume-to-capacity ratio" in this chapter refer to demand volume or demand-volume-to-capacity ratio.

LOS can be characterized for the entire intersection, each intersection approach, and each lane group. Control delay alone is used to characterize LOS for the entire intersection or an approach. Control delay *and* volume-to-capacity ratio are used to characterize LOS for a lane group. Delay quantifies the increase in travel time due to traffic signal control. It is also a surrogate measure of driver discomfort and fuel consumption. The volume-to-capacity ratio quantifies the degree to which a phase's capacity is utilized by a lane group. The following paragraphs describe each LOS.

LOS A describes operations with a control delay of 10 s/veh or less and a volume-to-capacity ratio no greater than 1.0. This level is typically assigned when the volume-to-capacity ratio is low and either progression is exceptionally favorable or the cycle length is very short. If it is due to favorable progression, most vehicles arrive during the green indication and travel through the intersection without stopping.

LOS B describes operations with control delay between 10 and 20 s/veh and a volume-to-capacity ratio no greater than 1.0. This level is typically assigned when the volume-to-capacity ratio is low and either progression is highly favorable or the cycle length is short. More vehicles stop than with LOS A.

LOS C describes operations with control delay between 20 and 35 s/veh and a volume-to-capacity ratio no greater than 1.0. This level is typically assigned when progression is favorable or the cycle length is moderate. Individual *cycle failures* (i.e., one or more queued vehicles are not able to depart as a result of insufficient capacity during the cycle) may begin to appear at this level. The number of vehicles stopping is significant, although many vehicles still pass through the intersection without stopping.

LOS D describes operations with control delay between 35 and 55 s/veh and a volume-to-capacity ratio no greater than 1.0. This level is typically assigned when the volume-to-capacity ratio is high and either progression is ineffective or the cycle length is long. Many vehicles stop and individual cycle failures are noticeable.

LOS E describes operations with control delay between 55 and 80 s/veh and a volume-to-capacity ratio no greater than 1.0. This level is typically assigned when the volume-to-capacity ratio is high, progression is unfavorable, and the cycle length is long. Individual cycle failures are frequent.

LOS F describes operations with control delay exceeding 80 s/veh or a volume-to-capacity ratio greater than 1.0. This level is typically assigned when the volume-to-capacity ratio is very high, progression is very poor, and the cycle length is long. Most cycles fail to clear the queue.

A lane group can incur a delay less than 80 s/veh when the volume-to-capacity ratio exceeds 1.0. This condition typically occurs when the cycle length is short, the signal progression is favorable, or both. As a result, both the delay and volume-to-capacity ratio are considered when lane group LOS is established. A ratio of 1.0 or more indicates that cycle capacity is fully utilized and represents failure from a capacity perspective (just as delay in excess of 80 s/veh represents failure from a delay perspective).

Exhibit 18-4 lists the LOS thresholds established for the automobile mode at a signalized intersection.

Exhibit 18-4
LOS Criteria: Automobile Mode

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

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

Nonautomobile Modes

Historically, the HCM has used a single performance measure as the basis for defining LOS. However, research documented in Chapter 5, Quality and Level-of-Service Concepts, indicates that travelers consider a wide variety of factors in assessing the quality of service provided to them. Some of these factors can be described as performance measures (e.g., speed) and others can be described as basic descriptors of the intersection character (e.g., crosswalk width). The methodology for evaluating each mode provides a procedure for mathematically combining these factors into a score. This score is then used to determine the LOS that is provided.

Exhibit 18-5 lists the range of scores associated with each LOS for the pedestrian and bicycle travel modes. The association between score value and LOS is based on traveler perception research. Travelers were asked to rate the quality of service associated with a specific trip through a signalized intersection. The letter A was used to represent the best quality of service, and the letter F was used to represent the worst quality of service. “Best” and “worst” were left undefined, allowing respondents to identify the best and worst conditions on the basis of their traveling experience and perception of service quality.

Exhibit 18-5
LOS Criteria: Pedestrian and Bicycle Modes

LOS	LOS Score
A	≤2.00
B	>2.00–2.75
C	>2.75–3.50
D	>3.50–4.25
E	>4.25–5.00
F	>5.00

REQUIRED INPUT DATA

This subsection describes the required input data for the automobile, pedestrian, and bicycle methodologies. Default values for some of these data are provided in Section 3, Applications.

Automobile Mode

This part describes the input data needed for the automobile methodology. The data needed for fully or semiactuated signal control are listed in Exhibit 18-6. The additional data needed for coordinated-actuated control are listed in Exhibit 18-7.

The last column of Exhibit 18-6 and Exhibit 18-7 indicates whether the input data are needed for each traffic movement, a specific movement group, each signal phase, each intersection approach, or the intersection as a whole.

The data elements listed in Exhibit 18-6 and Exhibit 18-7 do not include variables that are considered to represent calibration factors (e.g., start-up lost time). Default values are provided for these factors because they typically have a relatively narrow range of reasonable values or they have a small impact on the accuracy of the performance estimates. The recommended value for each calibration factor is identified at relevant points in the presentation of the methodology.

Exhibit 18-6
Input Data Requirements: Automobile Mode with Pretimed, Fully Actuated, or Semiactuated Signal Control

Data Category	Input Data Element	Basis
Traffic characteristics	Demand flow rate	Movement
	Right-turn-on-red flow rate	Approach
	Percent heavy vehicles	Movement group
	Intersection peak hour factor	Intersection
	Platoon ratio	Movement group
	Upstream filtering adjustment factor	Movement group
	Initial queue	Movement group
	Base saturation flow rate	Movement group
	Lane utilization adjustment factor	Movement group
	Pedestrian flow rate	Approach
	Bicycle flow rate	Approach
	On-street parking maneuver rate	Movement group
	Local bus stopping rate	Approach
Geometric design	Number of lanes	Movement group
	Average lane width	Movement group
	Number of receiving lanes	Approach
	Turn bay length	Movement group
	Presence of on-street parking	Movement group
	Approach grade	Approach
Signal control	Type of signal control	Intersection
	Phase sequence	Intersection
	Left-turn operational mode	Approach
	Dallas left-turn phasing option	Approach
	Passage time (if actuated)	Phase
	Maximum green (or green duration if pretimed)	Phase
	Minimum green	Phase
	Yellow change	Phase
	Red clearance	Phase
	Walk	Phase
	Pedestrian clear	Phase
	Phase recall	Phase
	Dual entry (if actuated)	Phase
	Simultaneous gap-out (if actuated)	Approach
Other	Analysis period duration	Intersection
	Speed limit	Approach
	Stop-line detector length and detection mode	Movement group
	Area type	Intersection

Notes: Movement = one value for each left-turn, through, and right-turn movement.
Movement group = one value for each turn movement with exclusive turn lanes and one value for the through movement (inclusive of any turn movements in a shared lane).
Approach = one value or condition for the intersection approach.
Intersection = one value or condition for the intersection.
Phase = one value or condition for each signal phase.

Exhibit 18-7
Input Data Requirements: Automobile Mode with Coordinated-Actuated Signal Control

Data Category	Input Data Element	Basis
Signal control	Cycle length	Intersection
	Phase splits	Phase
	Offset	Intersection
	Offset reference point	Intersection
	Force mode	Intersection

Notes: Intersection = one value or condition for the intersection.
Phase = one value or condition for each signal phase.

Traffic Characteristics Data

This subpart describes the traffic characteristics data listed in Exhibit 18-6. These data describe the motorized vehicle traffic stream that travels through the intersection during the study period.

Demand Flow Rate

The demand flow rate for an intersection traffic movement is defined as the count of vehicles arriving at the intersection during the analysis period divided by the analysis period duration. It is expressed as an hourly flow rate but may represent an analysis period shorter than 1 h. Demand flow rate represents the flow rate of vehicles *arriving* at the intersection. When measured in the field, this flow rate is based on a traffic count taken upstream of the queue associated with the subject intersection. This distinction is important for counts during congested periods because the count of vehicles departing from a congested approach will produce a demand flow rate that is lower than the true rate.

There is one exception to the aforementioned definition of demand flow rate. Specifically, if a planning analysis is being conducted where (*a*) the projected demand flow rate coincides with a 1-h period and (*b*) an analysis of the peak 15-min period is desired, then each movement's hourly demand can be divided by the intersection peak hour factor to predict the flow rate during the peak 15-min period. The peak hour factor should be based on local traffic peaking trends. If a local factor is not available, then the default value provided in Section 3 can be used.

In summary, demand flow rate for the analysis period is an input to the methodology. This rate is computed as the count of vehicles arriving during the period divided by the length of the period, expressed as an hourly flow rate, and without the use of a peak hour factor. If a peak hour factor is used, it must be used to compute the hourly flow rate that is input to the methodology.

If intersection operation is being evaluated during multiple sequential analysis periods, then the count of vehicles arriving during each analysis period should be provided for each movement.

The methodology includes a procedure for determining the distribution of flow among the available lanes on an approach with one or more shared lanes. The procedure is based on an assumed desire by drivers to choose the lane that minimizes their service time at the intersection, where the lane volume-to-saturation flow ratio is used to estimate relative differences in this time among lanes. This assumption may not always hold for situations in which drivers choose a lane so that they are prepositioned for a turn at the downstream intersection. In this situation, the analyst needs to provide the flow rate for each lane on the approach and then combine these rates to define explicitly the flow rate for each lane group.

Only right turns that are controlled by the signal should be represented in the right-turn volume input to the automobile methodology.

If a right-turn movement is allowed to turn right on the red indication, the analyst may reduce the right-turn flow rate by the flow rate of right-turn-on-red

(RTOR) vehicles. This topic is discussed in more detail in the next few paragraphs.

Right-Turn-on-Red Flow Rate

The RTOR flow rate is defined as the count of vehicles that turn right at the intersection when the controlling signal indication is red, divided by the analysis period duration. It is expressed as an hourly flow rate but may represent an analysis period shorter than 1 h.

It is difficult to predict the RTOR flow rate because it is based on many factors that vary widely from intersection to intersection. These factors include the following:

- Approach lane allocation (shared or exclusive right-turn lane),
- Right-turn flow rate,
- Sight distance available to right-turning drivers,
- Volume-to-capacity ratio for conflicting movements,
- Arrival patterns of right-turning vehicles during the signal cycle,
- Departure patterns of conflicting movements,
- Left-turn signal phasing on the conflicting street, and
- Conflicts with pedestrians.

Given the difficulty of estimating the RTOR flow rate, it should be measured in the field when possible. If the analysis is dealing with future conditions or if the RTOR flow rate is not known from field data, then the RTOR flow rate for each right-turn movement should be assumed to equal 0 veh/h. This assumption is conservative because it yields a slightly larger estimate of delay than may actually be incurred by intersection movements.

If the right-turn movement is served by an exclusive lane and a complementary left-turn phase exists on the cross street, then the right-turn volume for analysis can be reduced by the number of shadowed left turners (with both movements being considered on an equivalent, per lane basis).

Percent Heavy Vehicles

A heavy vehicle is defined as any vehicle with more than four tires touching the pavement. Local buses that stop within the intersection area are not included in the count of heavy vehicles. The percentage of heavy vehicles represents the count of heavy vehicles that arrive during the analysis period divided by the total vehicle count for the same period. This percentage is provided for each intersection traffic movement; however, one representative value for all movements may be used for a planning analysis.

Intersection Peak Hour Factor

One peak hour factor for the entire intersection is computed with the following equation:

$$PHF = \frac{n_{60}}{4\, n_{15}}$$

Equation 18-1

where

PHF = peak hour factor,

n_{60} = count of vehicles during a 1-h period (veh), and

n_{15} = count of vehicles during the peak 15-min period (veh).

The count used in the denominator of Equation 18-1 must be taken during a 15-min period that occurs within the 1-h period represented by the variable in the numerator. Both variables in this equation represent the total number of vehicles entering the intersection during their respective time period. As such, one peak hour factor is computed for the intersection. This factor is then applied individually to each traffic movement. Values of this factor typically range from 0.80 to 0.95.

As noted previously, the peak hour factor is used primarily for a planning analysis when a forecast hourly volume is provided and an analysis of the peak 15-min period is sought. Normally, the demand flow rate is computed as the count of vehicles arriving during the period divided by the length of the period, expressed as an hourly flow rate, and without the use of a peak hour factor.

The use of a single peak hour factor for the entire intersection is intended to avoid the likelihood of creating demand scenarios with conflicting volumes that are disproportionate to the actual volumes during the 15-min analysis period. If peak hour factors for each individual approach or movement are used, they are likely to generate demand volumes from one 15-min period that are in apparent conflict with demand volumes from another 15-min period, whereas in reality these peak volumes do not occur at the same time. Furthermore, to determine individual approach or movement peak hour factors, actual 15-min count data are likely available, permitting the determination of actual 15-min demand and avoiding the need to use a peak hour factor. In the event that individual approaches or movements are known to peak at different times, several 15-min analysis periods that encompass all the peaking should be considered instead of a single analysis in which all the peak hour factors are used together, as if the peaks they represent also occurred together.

Platoon Ratio

Platoon ratio is used to describe the quality of signal progression for the corresponding movement group. It is computed as the demand flow rate during the green indication divided by the average demand flow rate. Values for the platoon ratio typically range from 0.33 to 2.0. Exhibit 18-8 provides an indication of the quality of progression associated with selected platoon ratio values.

Exhibit 18-8
Relationship Between Arrival Type and Progression Quality

Platoon Ratio	Arrival Type	Progression Quality
0.33	1	Very poor
0.67	2	Unfavorable
1.00	3	Random arrivals
1.33	4	Favorable
1.67	5	Highly favorable
2.00	6	Exceptionally favorable

For protected or protected-permitted left-turn movements operating in an exclusive lane, platoon ratio is used to describe progression quality during the associated turn phase (i.e., the protected period). Hence, the platoon ratio is based on the flow rate during the green indication of the left-turn phase.

For permitted left-turn movements operating in an exclusive lane, platoon ratio is used to describe progression quality during the permitted period. Hence, the platoon ratio is based on the left-turn flow rate during the green indication of the phase providing the permitted operation.

For permitted or protected-permitted right-turn movements operating in an exclusive lane, platoon ratio is used to describe progression quality during the permitted period (even if a protected right-turn operation is provided during the complementary left-turn phase on the cross street). Hence, the platoon ratio is based on the right-turn flow rate during the green indication of the phase providing the permitted operation.

For through movements served by exclusive lanes (no shared lanes on the approach), the platoon ratio for the through movement group is based on the through flow rate during the green indication of the associated phase.

For all movements served by split phasing, the platoon ratio for a movement group is based on its flow rate during the green indication of the common phase.

For intersection approaches with one or more shared lanes, one platoon ratio is computed for the shared movement group on the basis of the flow rate of all shared lanes (plus that of any exclusive through lanes that are also served) during the green indication of the common phase.

The platoon ratio for a movement group can be estimated from field data with the following equation:

Equation 18-2

$$R_p = \frac{P}{(g/C)}$$

where

R_p = platoon ratio,

P = proportion of vehicles arriving during the green indication (decimal),

g = effective green time (s), and

C = cycle length (s).

The "proportion of vehicles arriving during the green indication" P is computed as the count of vehicles that arrive during the green indication divided by the count of vehicles that arrive during the entire signal cycle. It is an average value representing conditions during the analysis period.

If the subject intersection is part of a signal system, then the procedure in Chapter 17, Urban Street Segments, can be used to estimate the arrival flow profile for any approach that is evaluated as part of an urban street segment. The procedure uses the profile to compute the proportion of arrivals during the green indication. If this procedure is used, then platoon ratio is not an input for the traffic movements on the subject approach.

If the subject intersection is not part of a signal system and an existing intersection is being evaluated, then it is recommended that analysts use field-measured values for the variables in Equation 18-2 in estimating the platoon ratio.

If the subject intersection is not part of a signal system and the analysis is dealing with future conditions, or if the variables in Equation 18-2 are not known from field data, then the platoon ratio can be judged from Exhibit 18-8 by using the arrival type designation. Values of arrival type range from 1 to 6. A description of each arrival type is provided in the following paragraphs to help the analyst make a selection.

Arrival Type 1 is characterized by a dense platoon of more than 80% of the movement group volume arriving at the start of the red interval. This arrival type is often associated with short segments with very poor progression in the subject direction of travel (and possibly good progression for the other direction).

Arrival Type 2 is characterized by a moderately dense platoon arriving in the middle of the red interval or a dispersed platoon containing 40% to 80% of the movement group volume arriving throughout the red interval. This arrival type is often associated with segments of average length with unfavorable progression in the subject direction of travel.

Arrival Type 3 describes one of two conditions. If the signals bounding the segment are coordinated, then this arrival type is characterized by a platoon containing less than 40% of the movement group volume arriving partly during the red interval and partly during the green interval. If the signals are not coordinated, then this arrival type is characterized by platoons arriving at the subject intersection at different points in time over the course of the analysis period so that arrivals are effectively random.

Arrival Type 4 is characterized by a moderately dense platoon arriving in the middle of the green interval or a dispersed platoon containing 40% to 80% of the movement group volume arriving throughout the green interval. This arrival type is often associated with segments of average length with favorable progression in the subject direction of travel.

Arrival Type 5 is characterized by a dense platoon of more than 80% of the movement group volume arriving at the start of the green interval. This arrival type is often associated with short segments with highly favorable progression in the subject direction of travel and a low-to-moderate number of side street entries.

Arrival Type 6 is characterized by a dense platoon of more than 80% of the movement group volume arriving at the start of the green interval. This arrival type occurs only on very short segments with exceptionally favorable progression in the subject direction of travel and negligible side street entries. It is reserved for routes in dense signal networks, possibly with one-way streets.

Upstream Filtering Adjustment Factor

The upstream filtering adjustment factor I accounts for the effect of an upstream signal on vehicle arrivals to the subject movement group. Specifically, this factor reflects the way an upstream signal changes the variance in the number of arrivals per cycle. The variance decreases with increasing volume-to-capacity ratio, which can reduce cycle failure frequency and resulting delay.

The filtering adjustment factor varies from 0.09 to 1.0. A value of 1.0 is appropriate for an isolated intersection (i.e., one that is 0.6 mi or more from the nearest upstream signalized intersection). A value of less than 1.0 is appropriate for nonisolated intersections. The following equation is used to compute I for nonisolated intersections:

Equation 18-3

$$I = 1.0 - 0.91\ X_u^{2.68} \quad \geq 0.090$$

where

I = upstream filtering adjustment factor, and

X_u = weighted volume-to-capacity ratio for all upstream movements contributing to the volume in the subject movement group.

The variable X_u is computed as the weighted volume-to-capacity ratio of all upstream movements contributing to the volume in the subject movement group. This ratio is computed as a weighted average with the volume-to-capacity ratio of each contributing upstream movement weighted by its discharge volume. For planning and design analyses, X_u can be approximated as the volume-to-capacity ratio of the contributing through movement at the upstream signalized intersection. The value of X_u used in Equation 18-3 cannot exceed 1.0.

Initial Queue

The initial queue represents the queue present at the start of the subject analysis period for the subject movement group. This queue is created when oversaturation is sustained for an extended time. The initial queue can be estimated by monitoring queue count continuously during each of the three consecutive cycles that occur just before the start of the analysis period. The smallest count observed during each cycle is recorded. The initial queue estimate equals the average of the three counts. The initial queue estimate should not include vehicles in the queue due to random, cycle-by-cycle fluctuations.

Base Saturation Flow Rate

The saturation flow rate represents the maximum rate of flow for a traffic lane, as measured at the stop line during the green indication. The base saturation flow rate represents the saturation flow rate for a traffic lane that is 12 ft wide and has no heavy vehicles, a flat grade, no parking, no buses that stop at the intersection, even lane utilization, and no turning vehicles. Typically, one base rate is selected to represent all signalized intersections in the jurisdiction (or area) within which the subject intersection is located. It has units of passenger cars per hour per lane (pc/h/ln). Chapter 31, Signalized Intersections: Supplemental, describes a field measurement technique for quantifying the local base saturation flow rate.

Lane Utilization Adjustment Factor

The lane utilization adjustment factor accounts for the unequal distribution of traffic among the lanes in those movement groups with more than one exclusive lane. This factor provides an adjustment to the base saturation flow rate to account for uneven use of the lanes. It is not used unless a movement group has more than one exclusive lane. It is calculated with Equation 18-4.

$$f_{LU} = \frac{v_g}{N_e \, v_{g1}}$$

Equation 18-4

where

f_{LU} = adjustment factor for lane utilization,

v_g = demand flow rate for movement group (veh/h),

v_{g1} = demand flow rate in the single exclusive lane with the highest flow rate of all exclusive lanes in movement group (veh/h/ln), and

N_e = number of exclusive lanes in movement group (ln).

Lane flow rates measured in the field can be used with Equation 18-4 to establish local default values of the lane utilization adjustment factor.

A lane utilization factor of 1.0 is used when a uniform traffic distribution can be assumed across all exclusive lanes in the movement group or when a movement group has only one lane. Values less than 1.0 apply when traffic is not uniformly distributed. As demand approaches capacity, the lane utilization factor is often closer to 1.0 because drivers have less opportunity to select their lane.

At some intersections, drivers may choose one through lane over another lane in anticipation of a turn at a downstream intersection. When this type of “prepositioning” occurs, a more accurate evaluation will be obtained when the actual flow rate for each approach lane is measured in the field and provided as an input to the methodology.

Pedestrian Flow Rate

The pedestrian flow rate is based on the count of pedestrians traveling in the crosswalk that is crossed by vehicles turning right from the subject approach during the analysis period. For example, the pedestrian flow rate for the westbound approach describes the pedestrian flow in the crosswalk on the north leg. A separate count is taken for each direction of travel in the crosswalk. Each count is divided by the analysis period duration to yield a directional hourly flow rate. These rates are then added to obtain the pedestrian flow rate.

Bicycle Flow Rate

The bicycle flow rate is based on the count of bicycles whose travel path is crossed by vehicles turning right from the subject approach during the analysis period. These bicycles may travel on the shoulder or in a bike lane. Any bicycle traffic operating in the right lane with automobile traffic should not be included in this count. This interaction is not modeled by the methodology. The count is divided by the analysis period duration to yield an hourly flow rate.

On-Street Parking Maneuver Rate

The parking maneuver rate represents the count of *influential* parking maneuvers that occur on an intersection leg, as measured during the analysis period. An influential maneuver occurs directly adjacent to a movement group, within a zone that extends from the stop line to a point 250 ft upstream of it. A maneuver occurs when a vehicle enters or exits a parking stall. If more than 180 maneuvers/h exist, then a practical limit of 180 should be used. On a two-way leg, maneuvers are counted for just the right side of the leg. On a one-way leg, maneuvers are separately counted for each side of the leg. The count is divided by the analysis period duration to yield an hourly flow rate.

Local Bus Stopping Rate

The bus stopping rate represents the number of local buses that stop and block traffic flow in a movement group within 250 ft of the stop line (upstream or downstream), as measured during the analysis period. A *local bus* is a bus that stops to discharge or pick up passengers at a bus stop. The stop can be on the near side or the far side of the intersection. If more than 250 buses/h exist, then a practical limit of 250 should be used. The count is divided by the analysis period duration to yield an hourly flow rate.

Geometric Design Data

This subpart describes the geometric design data listed in Exhibit 18-6. These data describe the geometric elements of the intersection that influence traffic operation.

Number of Lanes

The number of lanes represents the count of lanes provided for each intersection traffic movement. For a turn movement, this count represents the lanes reserved for the exclusive use of turning vehicles. Turn movement lanes include turn lanes that extend backward for the length of the segment and lanes in a turn bay. Lanes that are shared by two or more movements are included in the count of through lanes and are described as *shared lanes*. If no exclusive turn lanes are provided, then the turn movement is indicated to have 0 lanes.

Average Lane Width

The average lane width represents the average width of the lanes represented in a movement group. The minimum average lane width is 8 ft. Standard lane widths are 12 ft. Lane widths greater than 16 ft can be included; however, the analyst should consider whether the wide lane actually operates as two narrow lanes. The analysis should reflect the way in which the lane width is actually used or expected to be used.

Number of Receiving Lanes

The number of receiving lanes represents the count of lanes departing the intersection. This number should be separately determined for each left-turn and right-turn movement. Experience indicates that proper turning cannot be executed at some intersections because a receiving lane is frequently blocked by double-parked vehicles. For this reason, the number of receiving lanes should be determined from field observation when possible.

Turn Bay Length

Turn bay length represents the length of the bay for which the lanes have full width and in which queued vehicles can be stored. Bay length is measured parallel to the roadway centerline. If there are multiple lanes in the bay and they have different lengths, then the length entered should be an average value.

If a two-way left-turn lane is provided for left-turn vehicle storage and adjacent access points exist, then the bay length entered should represent the "effective" storage length available to the left-turn movement. The determination of effective length is based on consideration of the adjacent access points and the associated left-turning vehicles that store in the two-way left-turn lane.

Presence of On-Street Parking

This input indicates whether on-street parking is allowed along the curb line adjacent to a movement group and within 250 ft upstream of the stop line during the analysis period. On a two-way street, the presence of parking is noted for just the right side of the street. On a one-way street, the presence of on-street parking is separately noted for each side of the street.

Approach Grade

Approach grade defines the average grade along the approach, as measured from the stop line to a point 100 ft upstream of the stop line along a line parallel to the direction of travel. An uphill condition has a positive grade, and a downhill condition has a negative grade.

Signal Control Data

This subpart describes the signal control data listed in Exhibit 18-6 and Exhibit 18-7. They are specific to an actuated traffic signal controller that is operated in a pretimed, semiactuated, fully actuated, or coordinated-actuated manner.

Type of Signal Control

The methodology is based on the operation of a fully actuated controller. However, semiactuated, pretimed, and coordinated-actuated control can be achieved through proper specification of the controller inputs.

Semiactuated control is achieved by using the following settings for nonactuated phases:

- Maximum green is set to an appropriate value, and
- Maximum recall is invoked.

An equivalent pretimed control is achieved by using the following two settings for each signal phase:

- Maximum green is set to its desired pretimed green interval duration, and
- Maximum recall is invoked.

Settings used for coordinated-actuated control are described later in this subpart and are used in Chapter 17.

The automobile methodology is based on the latest controller functions defined in the National Transportation Communications for ITS Protocol Standard 1202. It is incumbent on the analyst to become familiar with these functions and adapt them, if needed, to the functionality of the controller that is used at the subject intersection. Chapter 31 provides additional information about traffic signal controller operation.

Phase Sequence

In a broad context, phase sequence describes the sequence of service provided to each traffic movement. This definition is narrowed here to limit phase sequence to a description of the order in which the left-turn movements are served, relative to the through movements. The sequence options addressed in the methodology include no left-turn phase, leading left-turn phase, lagging left-turn phase, and split phasing.

Left-Turn Operational Mode

The left-turn operational mode describes how the left-turn movement is served by the controller. It can be described as permitted, protected, or protected-permitted.

Dallas Left-Turn Phasing Option

This option allows the left-turn movements to operate in the protected-permitted mode without causing a "yellow trap" safety concern. It effectively ties the left turn's permitted period signal indication to the opposing through movement signal indication. This phasing option is also used with a flashing yellow arrow left-turn signal display.

Passage Time

Passage time is the maximum amount of time one vehicle actuation can extend the green interval while green is displayed. It is input for each actuated signal phase. It is also referred to as vehicle interval, extension interval, extension, or unit extension.

Passage time values are typically based on detection zone length, detection zone location (relative to the stop line), number of lanes served by the phase, and vehicle speed. Longer passage times are often used with shorter detection zones, greater distance between the zone and stop line, fewer lanes, and slower speeds.

The objective in determining the passage time value is to make it large enough to ensure that all queued vehicles are served but not so large that it extends for randomly arriving traffic. On high-speed approaches, this objective is broadened to include not making the passage time so long that the phase frequently extends to its maximum setting (i.e., maxes out) so that safe phase termination is compromised.

Maximum Green

The maximum green setting defines the maximum amount of time that a green signal indication can be displayed in the presence of conflicting demand. Typical maximum green values for left-turn phases range from 15 to 30 s. Typical values for through phases serving the minor-street approach range from 20 to 40 s, and those for through phases serving the major-street approach range from 30 to 60 s.

For an operational analysis of pretimed operation, the maximum green setting for each phase should equal the desired green interval duration and the recall mode should be set to "maximum." These settings also apply to the major-street through-movement phases for semiactuated operation.

For an analysis of coordinated-actuated operation, the maximum green is disabled through the inhibit mode and the phase splits are used to determine the maximum length of the actuated phases.

Minimum Green

The minimum green setting represents the least amount of time a green signal indication is displayed when a signal phase is activated. Its duration is based on consideration of driver reaction time, queue size, and driver expectancy. Minimum green typically ranges from 4 to 15 s, with shorter values in this range used for phases serving turn movements and lower-volume through movements. For intersections without pedestrian push buttons, the minimum green setting may also need to be long enough to allow time for pedestrians to react to the signal indication and cross the street.

Yellow Change and Red Clearance

The yellow change and the red clearance settings are input for each signal phase. The yellow change interval is intended to alert a driver to the impending presentation of a red indication. It ranges from 3 to 6 s, with longer values in this range used with phases serving high-speed movements. The red clearance interval can be used to allow a brief time to elapse after the yellow indication, during which the signal heads associated with the ending phase and all conflicting phases display a red indication. If used, the red clearance interval is typically 1 or 2 s.

Walk

The walk interval is intended to give pedestrians adequate time to perceive the WALK indication and depart the curb before the pedestrian clear interval begins.

For an actuated or a noncoordinated phase, the walk interval is typically set at the minimum value needed for pedestrian perception and curb departure. Many agencies consider this value to be 7 s; however, some agencies use as little as 4 s. Longer walk durations should be considered in school zones and areas with large numbers of elderly pedestrians. In the methodology, it is assumed that the rest-in-walk mode is not enabled for actuated phases and noncoordinated phases.

For a pretimed phase, the walk interval is often set at a value equal to the green interval duration needed for vehicle service less the pedestrian clear setting (provided that the resulting interval exceeds the minimum time needed for pedestrian perception and curb departure).

For a coordinated phase, the controller is sometimes set to use a coordination mode that extends the walk interval for most of the green interval duration. This functionality is not explicitly modeled in the automobile methodology, but it can be approximated by setting the walk interval to a value equal to the phase split minus the sum of the pedestrian clear, yellow change, and red clearance intervals.

If the walk and pedestrian clear settings are provided for a phase, then it is assumed that a pedestrian signal head is also provided. If these settings are not used, then it is assumed that any pedestrian accommodation needed is provided in the minimum green setting.

Pedestrian Clear

The pedestrian clear interval (also referred to as the pedestrian change interval) is intended to provide time for pedestrians who depart the curb during the WALK indication to reach the opposite curb (or the median). Some agencies set the pedestrian clear equal to the "crossing time," where crossing time equals the curb-to-curb crossing distance divided by the pedestrian walking speed of 3.5 ft/s. Other agencies set the pedestrian clear equal to the crossing time less the vehicle change period (i.e., the combined yellow change and red clearance intervals). This choice depends on agency policy and practice. A flashing DON'T WALK indication is displayed during this interval.

Phase Recall

If used, recall causes the controller to place a call for a specified phase each time the controller is servicing a conflicting phase. It is input for each signal phase. Three types of recalls are modeled in the automobile methodology: minimum recall, maximum recall, and pedestrian recall.

Invoking minimum recall causes the controller to place a continuous call for vehicle service on the phase and then service the phase until its minimum green interval times out. The phase can be extended if actuations are received.

Invoking maximum recall causes the controller to place a continuous call for vehicle service on the phase. It results in presentation of the green indication for its maximum duration every cycle. Using maximum recall on all phases yields an equivalent pretimed operation.

Invoking pedestrian recall causes the controller to place a continuous call for pedestrian service on the phase and then service the phase for at least an amount of time equal to its walk and pedestrian clear intervals (longer if vehicle detections are received). Pedestrian recall is used for phases that have a high probability of pedestrian demand every cycle and no pedestrian detection.

Dual Entry

The entry mode is used in dual-ring operation to specify whether a phase is to be activated (green) even though it has not received a call for service. Two entry modes are possible: dual entry and single entry. This mode is input for each actuated signal phase.

A phase operating in dual entry is available to be called by the controller, even if no actuations have been received for this phase. A phase operating in single entry will be called only if actuations have been received.

During the timing of a cycle, a point is reached where the next phase (or phases) to be timed is on the other side of a barrier. At this point, the controller will check the phases in each ring and determine which phase to activate. If a call does not exist in a ring, the controller will activate a phase designated as dual entry in that ring. If two phases are designated as dual entry in the ring, then the first phase to occur in the phase sequence is activated.

Simultaneous Gap-Out

The simultaneous gap-out mode affects the way actuated phases are terminated before the barrier can be crossed to serve a conflicting call. This mode can be enabled or disabled. It is a phase-specific setting; however, it is typically set the same for all phases that serve the same street. This mode is input for each actuated signal phase.

Simultaneous gap-out dictates controller operation when a barrier must be crossed to serve the next call and one phase is active in each ring. If simultaneous gap-out is enabled, it requires that both phases reach a point of being committed to terminate (via gap out, max out, or force-off) at the same time. If one phase is able to terminate because it has gapped out, but the other phase is not able to terminate, then the gapped-out phase will reset its extension timer and restart the process of timing down to gap-out.

If the simultaneous gap-out feature is disabled, then each phase can reach a point of termination independently. In this situation, the first phase to commit to termination maintains its active status while waiting for the other phase to commit to termination. Regardless of which mode is in effect, the barrier is not crossed until both phases are committed to terminate.

Cycle Length (Coordinated-Actuated Operation)

Cycle length is the time elapsed between the endings of two sequential presentations of a coordinated phase green interval.

Phase Splits (Coordinated-Actuated Operation)

Each noncoordinated phase is provided a “split” time. This time represents the sum of the green, yellow change, and red clearance intervals for the phase. The rationale for determining the green interval duration varies among agencies; however, it is often related to the “optimum” pretimed green interval duration. Chapter 31 describes a procedure for determining pretimed phase duration.

Offset and Offset Reference Point (Coordinated-Actuated Operation)

The reference phase is specified to be one of the two coordinated phases (i.e., Phase 2 or 6). The offset entered in the controller represents the time that the reference phase begins (or ends) relative to the system master time zero. The offset must be specified as being referenced to the beginning, or the end, of the green interval of the reference phase. The offset reference point is typically the same at all intersections in a given signal system.

Force Mode (Coordinated-Actuated Operation)

This mode is a controller-specific setting. It is set to "fixed" or "floating." The controller calculates the phase force-off point for each noncoordinated phase on the basis of the force mode and the phase splits. When set to the fixed mode, each noncoordinated phase has its force-off point set at a fixed time in the cycle, relative to time zero on the system master. This operation allows unused split time to revert to the following phase. When set to the floating mode, each noncoordinated phase has its force-off point set at the split time after the phase first becomes active. This operation allows unused split time to revert to the coordinated phase (referred to as an "early return to green").

Other Data

This subpart describes the data listed in Exhibit 18-6 that are categorized as "other" data.

Analysis Period Duration

The analysis period is the time interval considered for the performance evaluation. It ranges from 15 min to 1 h, with longer durations in this range sometimes used for planning analyses. In general, the analyst should interpret the results from an analysis period of 1 h or more with caution because the adverse impact of short peaks in traffic demand may not be detected. Also, if the analysis period is other than 15 min, then the peak hour factor should not be used.

The methodology was developed to evaluate conditions in which queue spillback does not affect the performance of the subject intersection or any upstream intersection during the analysis period. If spillback affects intersection performance, the analyst should consider use of an alternative analysis tool that is able to model the effect of spillback conditions.

Operational Analysis. A 15-min analysis period should be used for operational analyses. This duration will accurately capture the adverse effects of demand peaks. Any 15-min period of interest can be evaluated with the methodology; however, a complete evaluation should always include an analysis of conditions during the 15-min period that experiences the highest traffic demand during a 24-h period.

If traffic demand exceeds capacity for a given 15-min analysis period, then a multiple-period analysis should be conducted. This type of analysis consists of an evaluation of several consecutive 15-min time periods. The periods analyzed would include an initial analysis period that has no initial queue, one or more periods in which demand exceeds capacity, and a final analysis period that has no residual queue.

When a multiple-period analysis is used, intersection performance measures are computed for each analysis period. Averaging performance measures across multiple analysis periods is not encouraged because it may obscure extreme values.

Planning Analysis. A 15-min analysis period is used for most planning analyses. However, hourly traffic demands are normally produced through the planning process. Thus, when 15-min forecast demands are not available for a 15-min analysis period, a peak hour factor must be used to estimate the 15-min demands for the analysis period. A 1-h analysis period can be used, if appropriate. Regardless of analysis period duration, a single-period analysis is typical for planning applications.

Speed Limit

Average running speed is used in the methodology to evaluate lane group performance. It is correlated with speed limit when speed limit reflects the environmental and geometric factors that influence driver speed choice. As such, speed limit represents a single input variable that can be used as a convenient way to estimate running speed while limiting the need for numerous environmental and geometric input data.

The convenience of using speed limit as an input variable comes with a caution—the analyst must not infer a cause-and-effect relationship between the input speed limit and the estimated running speed. More specifically, the computed change in performance resulting from a change in the input speed limit is not likely to be indicative of performance changes that will actually be realized. Research indicates that a change in speed limit has a proportionally smaller effect on the actual average speed (*24*).

The methodology is based on the assumption that the posted speed limit is (*a*) consistent with that found on other streets in the vicinity of the subject intersection and (*b*) consistent with agency policy regarding specification of speed limits. If it is known that the posted speed limit does not satisfy these assumptions, then the speed limit value that is input to the methodology should be adjusted so that it is consistent with the assumptions.

Stop-Line Detector Length and Detection Mode

The stop-line detector length represents the length of the detection zone used to extend the green indication. This detection zone is typically located near the stop line and may have a length of 40 ft or more. However, it can be located some distance upstream of the stop line and may be as short as 6 ft. The latter configuration typically requires a long minimum green or use of the controller's variable initial setting.

If a video-image vehicle detection system is used to provide stop-line detection, then the length that is input should reflect the physical length of roadway that is monitored by the video detection zone plus a length of 5 to 10 ft to account for the projection of the vehicle image into the plane of the pavement (with larger values in this range used for wider intersections).

Detection mode influences the duration of the actuation submitted to the controller by the detection unit. One of two modes can be used: presence or pulse. Presence mode is typically the default mode. It tends to provide more reliable intersection operation than pulse mode.

In the presence mode, the actuation starts with the vehicle arriving in the detection zone and ends with the vehicle leaving the detection zone. Thus, the time duration of the actuation depends on vehicle length, detection zone length, and vehicle speed.

The presence mode is typically used with long detection zones located at the stop line. The combination typically results in the need for a small passage time value. This characteristic is desirable because it tends to result in efficient queue service.

In the pulse mode, the actuation starts and ends with the vehicle arriving at the detector (actually, the actuation is a short "on" pulse of 0.10 to 0.15 s). This mode is not used as often as presence mode for intersection control.

Area Type

The area type input is used to indicate whether the intersection is in a central business district (CBD) type of environment. An intersection is considered to be in a CBD, or a similar type of area, when its characteristics include narrow street rights-of-way, frequent parking maneuvers, vehicle blockages, taxi and bus activity, small-radius turns, limited use of exclusive turn lanes, high pedestrian activity, dense population, and midblock curb cuts. The average saturation headway at intersections in areas with these characteristics is significantly longer than that found at intersections in areas that are less constrained and less visually intense.

Nonautomobile Modes

This part describes the input data needed for the pedestrian and bicycle methodologies. The data are listed in Exhibit 18-9 and are identified as "input data elements."

Exhibit 18-9 categorizes each input data element by travel mode methodology. The association between a data element and its travel mode is indicated by the provision of text in the corresponding cell of Exhibit 18-9. When text is provided in a cell, it indicates whether the data are needed for a traffic movement, signal phase, intersection approach, intersection leg, or intersection as a whole. A blank cell indicates that the data element is not an input for the corresponding travel mode.

The data elements listed in Exhibit 18-9 do not include variables that are considered to represent calibration factors. Default values are provided for these factors because they typically have a relatively narrow range of reasonable values or they have a small impact on the accuracy of the performance estimates. The recommended value for each calibration factor is identified at the relevant point during presentation of the methodology.

Exhibit 18-9
Input Data Requirements: Nonautomobile Modes

Data Category	Input Data Element	Pedestrian Mode [a]	Bicycle Mode [a]
Traffic characteristics	Demand flow rate of motorized vehicles	Movement	Approach
	Right-turn-on-red flow rate	Approach	
	Permitted left-turn flow rate	Movement	
	Midsegment 85th percentile speed	Approach	
	Pedestrian flow rate	Movement	
	Bicycle flow rate		Approach
	Proportion of on-street parking occupied		Approach
Geometric design	Street width		Approach
	Number of lanes	Leg	Approach
	Number of right-turn islands	Leg	
	Width of outside through lane		Approach
	Width of bicycle lane		Approach
	Width of paved outside shoulder (or parking lane)		Approach
	Total walkway width	Approach	
	Crosswalk width	Leg	
	Crosswalk length	Leg	
	Corner radius	Approach	
Signal control	Walk	Phase	
	Pedestrian clear	Phase	
	Rest in walk	Phase	
	Cycle length	Intersection	Intersection
	Yellow change	Phase	Phase
	Red clearance	Phase	Phase
	Duration of phase serving pedestrians and bicycles	Phase	Phase
	Pedestrian signal head presence	Phase	
Other	Analysis period duration [b]	Intersection	Intersection

Notes: [a] Movement = one value for each left-turn, through, and right-turn movement.
Approach = one value for the intersection approach.
Leg = one value for the intersection leg (approach plus departure sides).
Intersection = one value for the intersection.
Phase = one value or condition for each signal phase.
[b] Analysis period duration is as defined for Exhibit 18-6.

Traffic Characteristics Data

This subpart describes the traffic characteristics data listed in Exhibit 18-9. These data describe the traffic streams traveling through the intersection during the study period. The demand flow rate of motorized vehicles, RTOR flow rate, and bicycle flow rate were defined in the previous subsection for the automobile mode.

Permitted Left-Turn Flow Rate

The permitted left-turn flow rate is defined as the count of vehicles that turn left permissively, divided by the analysis period duration. It is expressed as an hourly flow rate but may represent an analysis period shorter than 1 h. A permitted left-turn movement can occur with either the permitted or the protected-permitted left-turn mode. For left-turn movements served by the permitted mode, the permitted left-turn flow rate is equal to the left-turn demand flow rate.

For left-turn movements served by the protected-permitted mode, the permitted left-turn flow rate should be measured in the field because its value is influenced by many factors. Section 3, Applications, describes a procedure that can be used to estimate a default flow rate if the analysis involves future conditions or if the permitted left-turn flow rate is not known from field data.

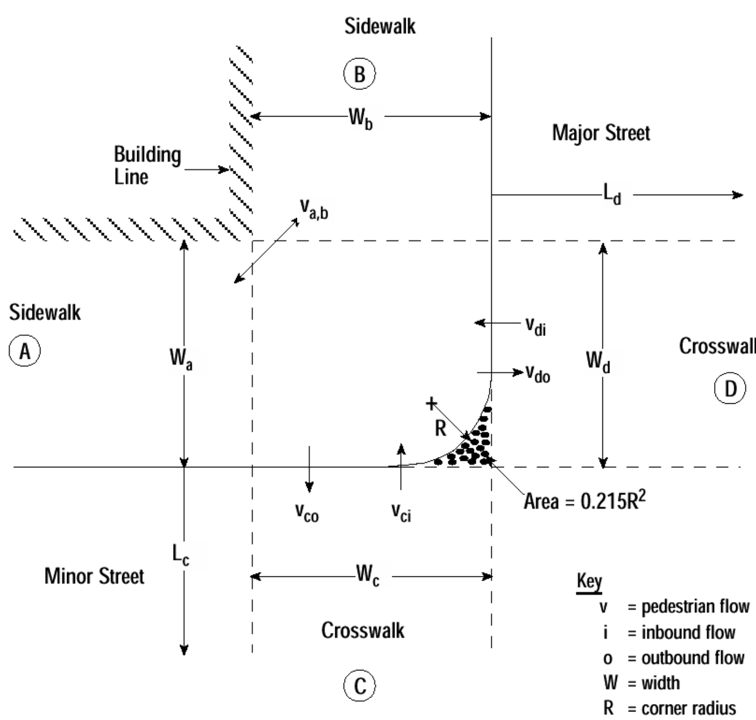
Midsegment 85th Percentile Speed

The 85th percentile speed represents the speed of the vehicle whose speed is exceeded by only 15% of the population of vehicles. The speed of interest is that of vehicles traveling along the street approaching the subject intersection. It is measured at a location sufficiently distant from the intersection that speed is not influenced by intersection operation. This speed is likely to be influenced by traffic conditions, so it should reflect the conditions present during the analysis period.

Pedestrian Flow Rate

The pedestrian flow rate represents the count of pedestrians traveling through each corner of the intersection divided by the analysis period duration. It is expressed as an hourly flow rate but may represent an analysis period shorter than 1 h. This flow rate is provided for each of five movements at each intersection corner. These five movements (i.e., v_{ci}, v_{co}, v_{di}, v_{do}, and $v_{a,b}$) are shown in Exhibit 18-10 as they occur at one intersection corner.

Exhibit 18-10
Intersection Corner Geometry and Pedestrian Movements



Proportion of On-Street Parking Occupied

This variable represents the proportion of the intersection's right-side curb line that has parked vehicles present during the analysis period. It is based on a zone that extends from a point 250 ft upstream of the intersection to the intersection, and a second zone that extends from the intersection to a point

250 ft downstream of the intersection. If parking is not allowed in these two zones, then this proportion equals 0.0.

Geometric Design Data

This subpart describes the geometric design data listed in Exhibit 18-9. These data describe the geometric elements that influence intersection performance from a pedestrian or bicyclist perspective. The number-of-lanes variable was defined in the previous subsection for the automobile mode.

Street Width

The street width represents the width of the cross street as measured along the outside through vehicle lane on the subject approach between the extended curb line limits of the cross street. It is measured for each intersection approach.

Width of Through Lane, Width of Bicycle Lane, and Width of Shoulder

Several individual elements of the cross section are described in this subpart. These elements include the width of the outside through vehicle lane, the bicycle lane adjacent to the outside lane, and the paved outside shoulder.

The width of each of these elements is mutually exclusive (i.e., not overlapped). The outside lane width does not include the width of the gutter.

Total Walkway Width, Crosswalk Width and Length, and Corner Radius

These geometric design data describe the pedestrian accommodations on each corner of the intersection. These data are shown in Exhibit 18-10. The total walkway width (i.e., W_a and W_b) is measured from the outside edge of the road pavement (or face of curb, if present) to the far edge of the sidewalk (as sometimes delineated by building face, fence, or landscaping).

The crosswalk width (i.e., W_c and W_d) represents an effective width. Unless there is a known width constraint, the crosswalk's effective width should be the same as its physical width. A width constraint may be found when vehicles are observed to encroach regularly into the crosswalk area or when an obstruction in the median (e.g., a signal pole or reduced-width cut in the median curb) narrows the walking space.

The crosswalk length (i.e., L_c and L_d) is measured from outside edge to outside edge of road pavement (or curb to curb, if present) along the marked pedestrian travel path.

Signal Control Data

This subpart describes the data in Exhibit 18-9 that are identified as "signal control." The walk, pedestrian clear, yellow change, and red clearance settings were defined in the previous subsection for the automobile mode.

Rest in Walk

A phase with the rest-in-walk mode enabled will dwell in walk as long as there are no conflicting calls. When a conflicting call is received, the pedestrian clear interval will time to its setting value before ending the phase. This mode can be enabled for any actuated phase. Signals that operate with coordinated-actuated operation may be set to use a coordination mode that enables the rest-in-walk mode. Typically, the rest-in-walk mode is not enabled. In this case, the walk and pedestrian clear intervals time to their respective setting values, and then the pedestrian signal indication dwells in a steady DON'T WALK indication until a conflicting call is received.

Cycle Length

Cycle length is predetermined for pretimed or coordinated-actuated control. Chapter 31 provides a procedure for estimating a reasonable cycle length for these two types of control when cycle length is unknown.

For semiactuated and fully actuated control, an average cycle length must be provided as input to use the pedestrian or bicycle methodologies. This length can be estimated by using the automobile methodology.

Pedestrian Signal Head Presence

The presence of a pedestrian signal head influences pedestrian crossing behavior. If a pedestrian signal head is provided, then pedestrians are assumed to use the crosswalk during the WALK and flashing DON'T WALK indications. If no pedestrian signal heads are provided, then pedestrians will cross during the green indication provided to vehicular traffic.

Duration of Phase Serving Pedestrians and Bicycles

The duration of each phase that serves a pedestrian or bicycle movement is a required input. This phase is typically the phase that serves the through movement that is adjacent to the sidewalk and for which the pedestrian, bicycle, and through vehicle travel paths are parallel. For example, Phases 2, 4, 6, and 8 are the phases serving the pedestrian and bicycle movements in Exhibit 18-3.

SCOPE OF THE METHODOLOGY

Three methodologies are presented in this chapter, one for each of the automobile, pedestrian, and bicycle modes. This subsection identifies the conditions for which each methodology applies.

- *Signalized intersections.* All methodologies can be used to evaluate intersection performance from the perspective of the corresponding travel mode. The automobile methodology is developed to replicate fully actuated controller operation. However, specific inputs to the methodology can be used to facilitate evaluation of coordinated-actuated, semiactuated, or pretimed control.
- *Steady flow conditions.* The three methodologies are based on the analysis of steady traffic conditions and, as such, are not well suited to the evaluation of unsteady conditions (e.g., congestion, queue spillback, signal preemption).
- *Target road users.* Collectively, the three methodologies were developed to estimate the LOS perceived by automobile drivers, pedestrians, and bicyclists. They were not developed to provide an estimate of the LOS perceived by other road users (e.g., commercial vehicle drivers,

automobile passengers, delivery truck drivers, recreational vehicle drivers). However, it is likely that the perceptions of these other road users are reasonably well represented by the road users for whom the methodologies were developed.

- *Target travel modes.* The automobile methodology addresses mixed automobile, motorcycle, truck, and transit traffic streams where the automobile represents the largest percentage of all vehicles. The pedestrian and bicycle methodologies address travel by walking and bicycle, respectively. The methodologies are not designed to evaluate the performance of other types of vehicles (e.g., golf carts, motorized bicycles).
- *Influences in the right-of-way.* A road user's perception of quality of service is influenced by many factors inside and outside the urban street right-of-way. However, the methodologies in this chapter were specifically constructed to exclude factors that are outside the right-of- way (e.g., buildings, parking lots, scenery, landscaped yards) that might influence a traveler's perspective. This approach was followed because factors outside the right-of-way are not under the direct control of the agency operating the street.
- *"Typical pedestrian" focus for pedestrian methodology.* The pedestrian methodology is not designed to reflect the perceptions of any particular pedestrian subgroup, such as pedestrians with disabilities. As such, the performance measures obtained from the methodology are not intended to be indicators of a sidewalk's compliance with U.S. Access Board guidelines related to Americans with Disabilities Act (ADA) requirements. For this reason, they should not be considered as a substitute for an ADA compliance assessment of a pedestrian facility.

LIMITATIONS OF THE METHODOLOGY

In general, the methodologies described in this chapter can be used to evaluate the performance of most traffic streams traveling through an intersection. However, the methodologies do not address all traffic conditions or intersection configurations. The inability to replicate the influence of a condition or configuration in the methodology represents a limitation. This subsection identifies the known limitations of the methodologies described in this chapter. If one or more of these limitations is believed to have an important influence on the performance of a specific intersection, then the analyst should consider the use of alternative methods or tools.

Automobile Mode

The automobile methodology does not explicitly account for the effect of the following conditions on intersection operation:

- Turn bay overflow;
- Multiple advance detectors in the same lane;
- Demand starvation due to a closely spaced upstream intersection;

- Queue spillback into the subject intersection from a downstream intersection;
- Queue spillback from the subject intersection into an upstream intersection;
- Premature phase termination due to short detection length, passage time, or both;
- RTOR volume prediction or resulting right-turn delay;
- Turn movements served by more than two exclusive lanes;
- A right-turn movement that is not under signal control;
- Through lane (or lanes) added just upstream of the intersection or dropped just downstream of the intersection; and
- Storage of shared-lane left-turning vehicles within the intersection to permit bypass by through vehicles in the same lane.

In addition to the above conditions, the methodology does not directly account for the following controller functions:

- Rest-in-walk mode for actuated and noncoordinated phases,
- Preemption or priority modes,
- Phase overlap, and
- Gap reduction or variable initial settings for actuated phases.

Nonautomobile Modes

This part identifies the limitations of the pedestrian and bicycle methodologies. These methodologies are not able to model the conditions offered in the following list:

- Presence of grades in excess of 2%, and
- Presence of railroad crossings.

In addition, the pedestrian methodology does not model the following conditions:

- Unpaved sidewalk, and
- Free (i.e., uncontrolled) channelized right turn with multiple lanes or high-speed operation.

2. METHODOLOGY

OVERVIEW

This section describes three methodologies for evaluating the performance of a signalized intersection. Each methodology addresses one possible travel mode through the intersection. Analysts should choose the combination of methodologies that are appropriate for their analysis needs.

A complete evaluation of intersection operation includes the separate examination of performance for all relevant travel modes. The performance measures associated with each mode are assessed independently of one another. They are not mathematically combined into a single indicator of intersection performance. This approach ensures that all performance impacts are considered on a mode-by-mode basis.

The focus of each methodology in this chapter is the signalized intersection. Chapter 17, Urban Street Segments, provides a methodology for quantifying the performance of an urban street segment. The methodology described in Chapter 16, Urban Street Facilities, can be used to combine the performance measures (for a specified travel mode) on successive segments into an overall measure of facility performance for that mode.

AUTOMOBILE MODE

This subsection provides an overview of the methodology for evaluating signalized intersection performance from the motorist perspective. The methodology is computationally intense and requires software to implement. The intensity stems partly from the need to model traffic-actuated signal operation. Default values are provided in Section 3, Applications, to support planning analyses for which the required input data are not available.

A quick estimation method for evaluating intersection performance at a planning level of analysis is provided in Chapter 31, Signalized Intersections: Supplemental. This method is not computationally intense and can be applied by using hand calculations.

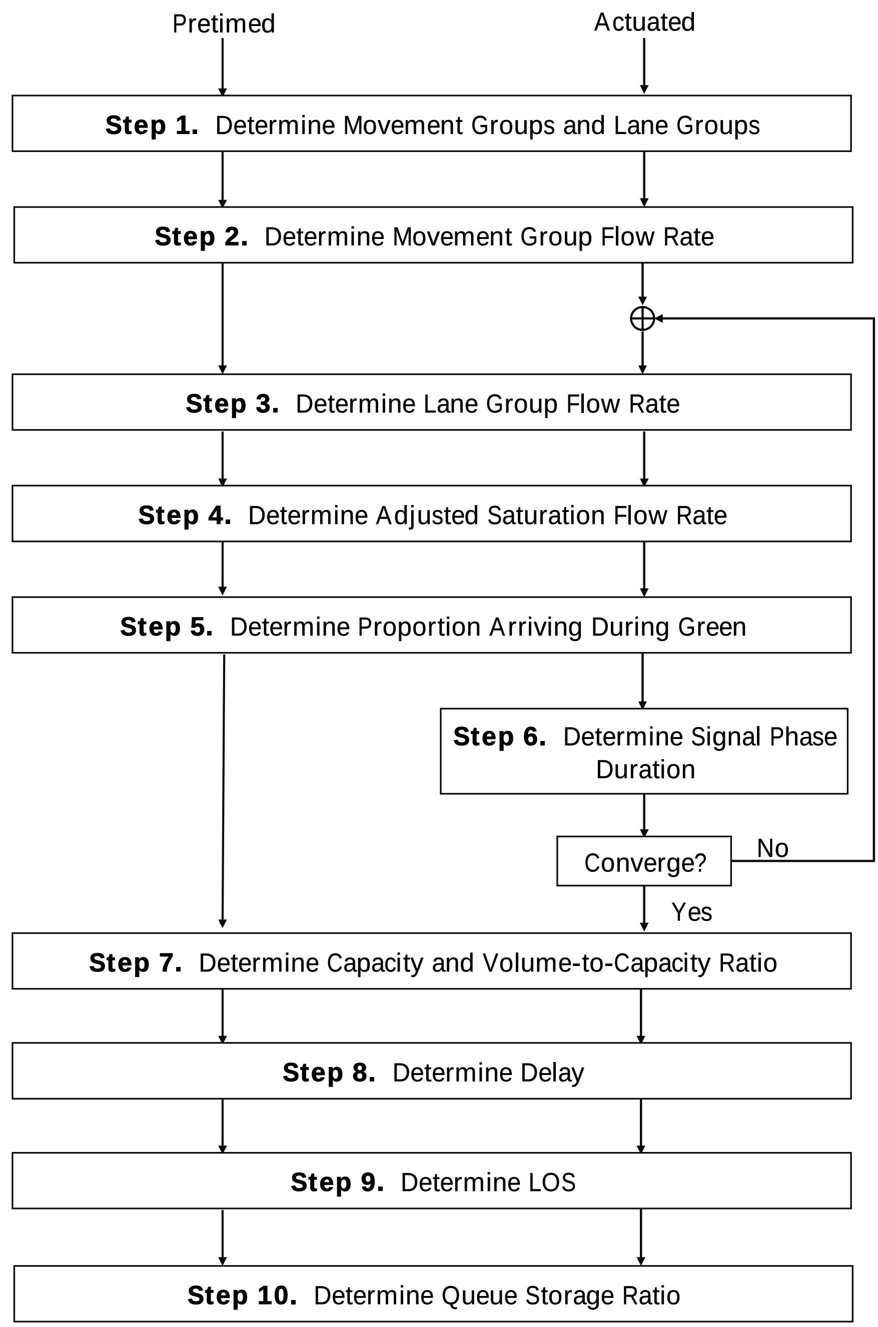
Because of the intensity of the computations, the objective of this subsection is to introduce the analyst to the calculation process and discuss the key analytic procedures. This objective is achieved by focusing the discussion on lane groups that serve one traffic movement with pretimed control and for which there are no permitted or protected-permitted left-turn movements. Details on evaluation of actuated control, shared-lane lane groups, and intersections with permitted or protected-permitted left-turn operation are provided in Chapter 31.

The computational engine developed by the Transportation Research Board Committee on Highway Capacity and Quality of Service represents the most detailed description of this methodology. Additional information about this engine is provided in Chapter 31.

Framework

Exhibit 18-11 illustrates the calculation framework of the automobile methodology. It identifies the sequence of calculations needed to estimate selected performance measures. The calculation process is shown to flow from top to bottom in the exhibit. These calculations are described more fully in the remainder of this subsection.

Exhibit 18-11
Automobile Methodology for Signalized Intersections



Step 1: Determine Movement Groups and Lane Groups

The methodology for signalized intersections uses the concept of *movement groups* and *lane groups* to describe and evaluate intersection operation. These two group designations are very similar in meaning. In fact, their differences emerge only when a shared lane is present on an approach with two or more lanes. Each designation is defined in the following paragraphs. The movement-group designation is a useful construct for specifying input data. In contrast, the lane-group designation is a useful construct for describing the calculations associated with the methodology.

The following rules are used to determine movement groups for an intersection approach:

- A turn movement that is served by one or more exclusive lanes and no shared lanes should be designated as a movement group.
- Any lanes not assigned to a group by the previous rule should be combined into one movement group.

These rules result in the designation of one to three movement groups for each approach.

The concept of lane groups is useful when a shared lane is present on an approach that has two or more lanes. Several procedures in the methodology require some indication of whether the shared lane serves a mix of vehicles or functions as an exclusive turn lane. This issue cannot be resolved until the proportion of turns in the shared lane has been computed. If the computed proportion of turns in the shared lane equals 1.0 (i.e., 100%), the shared lane is considered to operate as an exclusive turn lane.

The following rules are used to determine lane groups for an intersection approach:

- An exclusive left-turn lane or lanes should be designated as a separate lane group. The same is true of an exclusive right-turn lane.
- Any shared lane should be designated as a separate lane group.
- Any lanes that are not exclusive turn lanes or shared lanes should be combined into one lane group.

These rules result in the designation of one or more of the following lane group possibilities for an intersection approach:

- Exclusive left-turn lane (or lanes),
- Exclusive through lane (or lanes),
- Exclusive right-turn lane (or lanes),
- Shared left-turn and through lane,
- Shared left-turn and right-turn lane,
- Shared right-turn and through lane, and
- Shared left-turn, through, and right-turn lane.

The methodology can be applied to any logical combination of these lane groups. Exhibit 18-12 shows some common movement groups and lane groups.

Exhibit 18-12
Typical Lane Groups for Analysis

Number of Lanes	Movements by Lanes	Movement Groups (MG)	Lane Groups (LG)
1	Left, thru., & right:	MG 1:	LG 1:
2	Exclusive left: Thru. & right:	MG 1: MG 2:	LG 1: LG 2:
2	Left & thru.: Thru. & right:	MG 1:	LG 1: LG 2:
3	Exclusive left: Exclusive left: Through: Through: Thru. & right:	MG 1: MG 2:	LG 1: LG 2: LG 3:

Step 2: Determine Movement Group Flow Rate

The flow rate for each movement group is determined in this step. If a turn movement is served by one or more exclusive lanes and no shared lanes, then that movement's flow rate is assigned to a movement group. Any of the approach flow that is yet to be assigned to a movement group (following application of the guidance in the previous sentence) is assigned to one movement group.

The RTOR flow rate is subtracted from the right-turn flow rate, regardless of whether the right turn occurs from a shared or an exclusive lane. At an existing intersection, the number of RTORs should be determined by field observation.

Step 3: Determine Lane Group Flow Rate

The lane group flow rate is determined in this step. If there are no shared lanes on the intersection approach or the approach has only one lane, there is a one-to-one correspondence between lane groups and movement groups. In this situation, the lane group flow rate equals the movement group flow rate.

If there are one or more shared lanes on the approach and two or more lanes, then the lane group flow rate is computed by the procedure described in Chapter 31. This procedure is based on an assumed desire by drivers to choose the lane that minimizes their service time at the intersection, where the lane volume-to-saturation flow ratio is used to estimate relative differences in this time among lanes. This assumption may not always hold for situations in which drivers choose a lane on the subject approach so that they are prepositioned for a turn at a downstream intersection. In this situation, the analyst needs to provide as input the demand flow rate for each lane on the approach and aggregate them as appropriate to define the lane group flow rate.

Step 4: Determine Adjusted Saturation Flow Rate

The adjusted saturation flow rate for each lane of each lane group is computed in this step. The base saturation flow rate provided as an input variable is used in this computation.

The computed saturation flow rate is referred to as the "adjusted" saturation flow rate because it reflects the application of various factors that adjust the base saturation flow rate to the specific conditions present on the subject intersection approach.

The procedure described in this step applies to lane groups that consist of an exclusive lane (or lanes) operating in a pretimed protected mode and without pedestrian or bicycle interaction. When these conditions do not hold, the supplemental procedures described in Chapter 31 should be combined with those in this step to compute the adjusted saturation flow rate.

Equation 18-5 is used to compute the adjusted saturation flow rate per lane for the subject lane group:

$$s = s_o \, f_w \, f_{HV} \, f_g \, f_p \, f_{bb} \, f_a \, f_{LU} \, f_{LT} \, f_{RT} \, f_{Lpb} \, f_{Rpb}$$

Equation 18-5

where

s = adjusted saturation flow rate (veh/h/ln),

s_o = base saturation flow rate (pc/h/ln),

f_w = adjustment factor for lane width,

f_{HV} = adjustment factor for heavy vehicles in traffic stream,

f_g = adjustment factor for approach grade,

f_p = adjustment factor for existence of a parking lane and parking activity adjacent to lane group,

f_{bb} = adjustment factor for blocking effect of local buses that stop within intersection area,

f_a = adjustment factor for area type,

f_{LU} = adjustment factor for lane utilization,

f_{LT} = adjustment factor for left-turn vehicle presence in a lane group,

f_{RT} = adjustment factor for right-turn vehicle presence in a lane group,

f_{Lpb} = pedestrian adjustment factor for left-turn groups, and

f_{Rpb} = pedestrian–bicycle adjustment factor for right-turn groups.

The adjustment factors in the list above are described in the following subparts.

Base Saturation Flow Rate

Computations begin with selection of a base saturation flow rate. This base rate represents the expected average flow rate for a through-traffic lane having geometric and traffic conditions that correspond to a value of 1.0 for each adjustment factor. Typically, one base rate is selected to represent all signalized intersections in the jurisdiction (or area) within which the subject intersection is located. Default values for this rate are provided in Section 3, Applications.

Adjustment for Lane Width

The lane width adjustment factor f_w accounts for the negative impact of narrow lanes on saturation flow rate and allows for an increased flow rate on wide lanes. Values of this factor are listed in Exhibit 18-13.

Exhibit 18-13 Lane Width Adjustment Factor

Average Lane Width (ft)	Adjustment Factor (f_w)
<10.0[a]	0.96
≥10.0–12.9	1.00
>12.9	1.04

Note: [a] Factors apply to average lane widths of 8.0 ft or more.

Standard lanes are 12 ft wide. The lane width factor may be used with caution for lane widths greater than 16 ft, or an analysis with two narrow lanes may be conducted. Use of two narrow lanes will always result in a higher saturation flow rate than a single wide lane, but, in either case, the analysis should reflect the way the width is actually used or expected to be used. In no case should this factor be used to estimate the saturation flow rate of a lane group with an average lane width that is less than 8.0 ft.

Adjustment for Heavy Vehicles

The heavy-vehicle adjustment factor f_{HV} accounts for the additional space occupied by heavy vehicles and for the difference in their operating capabilities, compared with passenger cars. This factor does not address local buses that stop in the intersection area. Values of this factor are computed with Equation 18-6.

Equation 18-6

$$f_{HV} = \frac{100}{100 + P_{HV}(E_T - 1)}$$

where

P_{HV} = percent heavy vehicles in the corresponding movement group (%), and

E_T = equivalent number of through cars for each heavy vehicle = 2.0.

Adjustment for Grade

The grade adjustment factor f_g accounts for the effects of approach grade on vehicle performance. Values of this factor are computed with Equation 18-7.

Equation 18-7

$$f_g = 1 - \frac{P_g}{200}$$

where P_g is the approach grade for the corresponding movement group (%).

This factor applies to grades ranging from –6.0% to +10.0%. An uphill grade has a positive value and a downhill grade has a negative value.

Adjustment for Parking

The parking adjustment factor f_p accounts for the frictional effect of a parking lane on flow in the lane group adjacent to the parking lane. It also accounts for the occasional blocking of an adjacent lane by vehicles moving into and out of parking spaces. If no parking is present, then this factor has a value of 1.00. If parking is present, then the value of this factor is computed with Equation 18-8.

$$f_p = \frac{N - 0.1 - \frac{18N_m}{3{,}600}}{N} \geq 0.050$$

Equation 18-8

where

N_m = parking maneuver rate adjacent to lane group (maneuvers/h), and

N = number of lanes in lane group (ln).

The parking maneuver rate corresponds to parking areas directly adjacent to the lane group and within 250 ft upstream of the stop line. A practical upper limit of 180 maneuvers/h should be maintained with Equation 18-8. A minimum value of f_p from this equation is 0.050. Each maneuver (either in or out) is assumed to block traffic in the lane next to the parking maneuver for an average of 18 s.

The factor applies only to the lane group that is adjacent to the parking. On a one-way street with a single-lane lane group, the number of maneuvers used is the total for both sides of the lane group. On a one-way street with two or more lane groups, the factor is calculated separately for each lane group and is based on the number of maneuvers adjacent to the group. Parking conditions with zero maneuvers have an impact different from that of a no-parking situation.

Adjustment for Bus Blockage

The bus-blockage adjustment factor f_{bb} accounts for the impact of local transit buses that stop to discharge or pick up passengers at a near-side or far-side bus stop within 250 ft of the stop line (upstream or downstream). Values of this factor are computed with Equation 18-9.

$$f_{bb} = \frac{N - \frac{14.4N_b}{3{,}600}}{N} \geq 0.050$$

Equation 18-9

where N is the number of lanes in lane group (ln) and N_b is the bus stopping rate on the subject approach (buses/h).

This factor should be used only when stopping buses block traffic flow in the subject lane group. A practical upper limit of 250 buses/h should be maintained with Equation 18-9. A minimum value of f_{bb} from this equation is 0.050. The factor used here assumes an average blockage time of 14.4 s during a green indication.

Adjustment for Area Type

The area type adjustment factor f_a accounts for the inefficiency of intersections in CBDs relative to those in other locations. When used, it has a value of 0.90.

Use of this factor should be determined on a case-by-case basis. This factor is not limited to designated CBD areas, nor does it need to be used for all CBD areas. Instead, it should be used in areas where the geometric design and the traffic or pedestrian flows, or both, are such that the vehicle headways are significantly increased.

Adjustment for Lane Utilization

The input lane utilization adjustment factor is used to estimate saturation flow rate for a lane group with more than one exclusive lane. If the lane group has one shared lane or one exclusive lane, then this factor is 1.0.

Adjustment for Right Turns

The right-turn adjustment factor f_{RT} is intended primarily to reflect the effect of right-turn path geometry on saturation flow rate. The value of this adjustment factor is computed with Equation 18-10.

Equation 18-10

$$f_{RT} = \frac{1}{E_R}$$

where E_R is the equivalent number of through cars for a protected right-turning vehicle (= 1.18).

If the right-turn movement shares a lane with another movement or has permitted operation, then the procedure described in Chapter 31 should be used to compute the adjusted saturation flow rate for the shared-lane lane group. The effect of pedestrians and bicycles on right-turn saturation flow rate is considered in a separate adjustment factor.

Adjustment for Left Turns

The left-turn adjustment factor f_{LT} is intended primarily to reflect the effect of left-turn path geometry on saturation flow rate. The value of this adjustment factor is computed with Equation 18-11.

Equation 18-11

$$f_{LT} = \frac{1}{E_L}$$

where E_L is the equivalent number of through cars for a protected left-turning vehicle (= 1.05).

If the left-turn movement shares a lane with another movement or has permitted operation, then the procedure described in Chapter 31 should be used to compute the adjusted saturation flow rate for the shared-lane lane group. The effect of pedestrians on left-turn saturation flow rate is considered in a separate adjustment factor.

Adjustment for Pedestrians and Bicycles

The procedure to determine the left-turn pedestrian–bicycle adjustment factor f_{Lpb} and the right-turn pedestrian–bicycle adjustment factor f_{Rpb} is based on the concept of conflict zone occupancy, which accounts for the conflict between turning vehicles, pedestrians, and bicycles. Relevant conflict zone occupancy takes into account whether the opposing vehicle flow is also in conflict with the left-turn movement. The proportion of green time in which the conflict zone is occupied is determined as a function of the relevant occupancy and the number of receiving lanes for the turning vehicles. A procedure for computing these factors is provided in Chapter 31.

Step 5: Determine Proportion Arriving During Green

Control delay and queue size at a signalized intersection are highly dependent on the proportion of vehicles that arrive during the green and red signal indications. Delay and queue size are smaller when a larger proportion of vehicles arrive during the green indication. Equation 18-12 is used to compute this proportion for each lane group.

$$P = R_p (g / C)$$

Equation 18-12

All variables are as previously defined. This equation requires knowledge of the effective green time *g* and cycle length *C*. These values are known for pretimed operation. If the intersection is not pretimed, then the average phase time and cycle length must be calculated by the procedures described in the next step.

The procedure in Chapter 17 can be used to estimate the arrival flow profile for an intersection approach when this approach is evaluated as part of an urban street segment. The procedure uses the profile to compute the proportion of arrivals during the green indication.

Step 6: Determine Signal Phase Duration

The duration of a signal phase depends on the type of control used at the subject intersection. If the intersection has pretimed control, then the phase duration is an input and this step is skipped. If the phase duration is unknown, then the pretimed phase duration procedure in Section 2 of Chapter 31 can be used to estimate the pretimed phase duration.

If the intersection has actuated control, then the actuated phase duration procedure in Section 2 of Chapter 31 is used in this step to estimate the average duration of an actuated phase. It distinguishes between actuated, noncoordinated, and coordinated phase types.

It is useful at this point to define the various terms that define phase duration. Some terms are specific to actuated operation; however, most constructs are equally applicable to pretimed operation.

The duration of an actuated phase is composed of five time periods. The first period represents the time lost while the queue reacts to the signal indication changing to green. The second interval represents the time required to clear the queue of vehicles. The third period represents the time the green indication is extended by randomly arriving vehicles. It ends when there is a gap in traffic (i.e., gap out) or the green extends to the maximum limit (i.e., max out). The fourth period represents the yellow change interval, and the fifth period represents the red clearance interval. The duration of an actuated phase is defined by Equation 18-13.

$$D_p = l_1 + g_s + g_e + Y + R_c$$

Equation 18-13

where

D_p = phase duration (s),

l_1 = start-up lost time = 2.0 (s),

g_s = queue service time (s),

g_e = green extension time (s),

Y = yellow change interval (s), and

R_c = red clearance interval (s).

The relationship between the variables in Equation 18-13 is shown in Exhibit 18-14 by using a queue accumulation polygon.

Exhibit 18-14
Time Elements Influencing Actuated Phase Duration

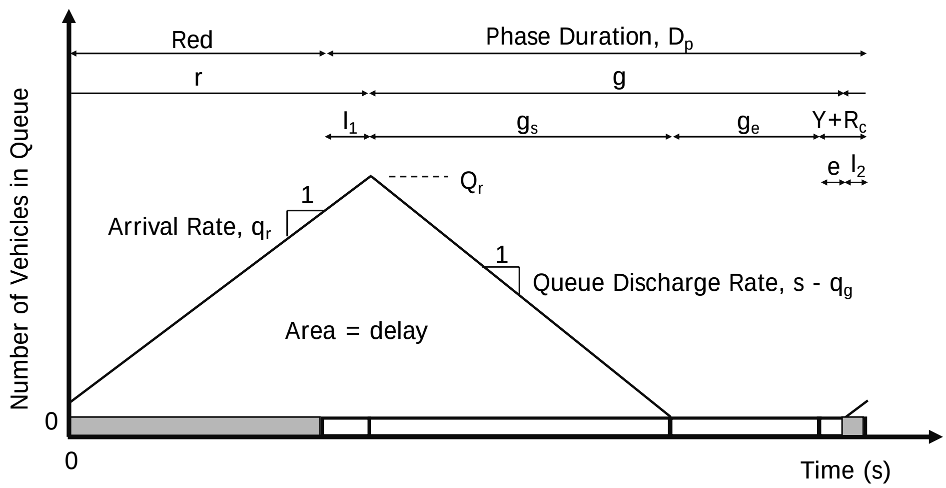


Exhibit 18-14 shows the relationship between phase duration and queue size for the average signal cycle. During the red interval, vehicles arrive at a rate of q_r and form a queue. The queue reaches its maximum size l_1 seconds after the red interval ends. At this time, the queue begins to discharge at a rate equal to the saturation flow rate s less the arrival rate during green q_g. The queue clears g_s seconds after it first begins to discharge. Thereafter, random vehicle arrivals are detected and cause the green interval to be extended. Eventually, a gap occurs in traffic (or the maximum green limit is reached) and the green interval ends. The end of the green interval coincides with the end of the extension time g_e.

The effective green time for the phase is computed with the following equation:

Equation 18-14

$$g = D_p - l_1 - l_2 = g_s + g_e + e$$

where

l_2 = clearance lost time = $Y + R_c - e$ (s),

e = extension of effective green = 2.0 (s), and

all other variables are as previously defined.

Step 7: Determine Capacity and Volume-to-Capacity Ratio

Lane Group Volume-to-Capacity Ratio

The capacity of a given lane group serving one traffic movement, and for which there are no permitted left-turn movements, is defined by Equation 18-15.

$$c = N\, s \frac{g}{C}$$

Equation 18-15

where c is the capacity (veh/h) and other variables are as previously defined. This equation cannot be used to calculate the capacity of a shared-lane lane group or a lane group with permitted left-turn operation because these lane groups have other factors that affect their capacity. Chapter 31 provides a procedure for estimating the capacity of these types of lane groups.

The volume-to-capacity ratio for a lane group is defined as the ratio of the lane group volume and its capacity. It is computed using Equation 18-16.

$$X = \frac{v}{c}$$

Equation 18-16

where

X = volume-to-capacity ratio,

v = demand flow rate (veh/h), and

c = capacity (veh/h).

Critical Intersection Volume-to-Capacity Ratio

Another concept used for analyzing signalized intersections is the critical volume-to-capacity ratio X_c. This ratio is computed by using Equation 18-17 with Equation 18-18.

$$X_c = \left(\frac{C}{C-L}\right) \sum_{i \in ci} y_{c,i}$$

Equation 18-17

with

$$L = \sum_{i \in ci} l_{t,i}$$

Equation 18-18

where

X_c = critical intersection volume-to-capacity ratio,

C = cycle length (s),

$y_{c,i}$ = critical flow ratio for phase $i = v_i/(Ns_i)$,

$l_{t,i}$ = phase i lost time = $l_{1,i} + l_{2,i}$ (s),

ci = set of critical phases on the critical path, and

L = cycle lost time (s).

The summation term in each of these equations represents the sum of a specific variable for the set of critical phases. A critical phase is one phase of a set of phases that occur in sequence and whose combined flow ratio is the largest for the signal cycle. The critical path and critical phases are identified by mapping traffic movements to a dual-ring phase diagram, as shown in Exhibit 18-3.

Equation 18-17 is based on the assumption that each critical phase has the same volume-to-capacity ratio and that this ratio is equal to the critical intersection volume-to-capacity ratio. This assumption is valid when the effective green duration for each critical phase i is proportional to $y_{c,i}/\Sigma(y_{c,i})$. When this assumption holds, the volume-to-capacity ratio for each noncritical phase is less than or equal to the critical intersection volume-to-capacity ratio.

Identifying Critical Lane Groups and Critical Flow Ratios

Calculation of the critical intersection volume-to-capacity ratio requires identification of the critical phases. This identification begins by mapping all traffic movements to a dual-ring diagram.

Next, the lane group flow ratio is computed for each lane group served by the phase. If a lane group is served only during one pretimed phase, then its flow ratio is computed as the lane group flow rate (per lane) divided by the lane group saturation flow rate [i.e., $v_i/(Ns_i)$]. If a lane group is served during multiple pretimed phases, then a flow ratio is computed for each phase. Specifically, the demand flow rate and saturation flow rate that occur during a given phase are used to compute the lane group flow ratio for that phase. For actuated phases, the flow ratio is computed only for those lane group-and-phase combinations in which the group's detectors actively extend the phase.

Next, the phase flow ratio is determined from the flow ratio of each lane group served during the phase. The phase flow ratio represents the largest flow ratio of all lane groups served.

Next, the diagram is evaluated to identify the critical phases. The phases that occur between one barrier pair are collectively evaluated to determine the critical phases. This evaluation begins with the pair in Ring 1 and proceeds to the pair in Ring 2. Each ring represents one possible critical path. The phase flow ratios are added for each phase pair in each ring. The larger of the two ring totals represents the critical path, and the corresponding phases represent the critical phases for the barrier pair.

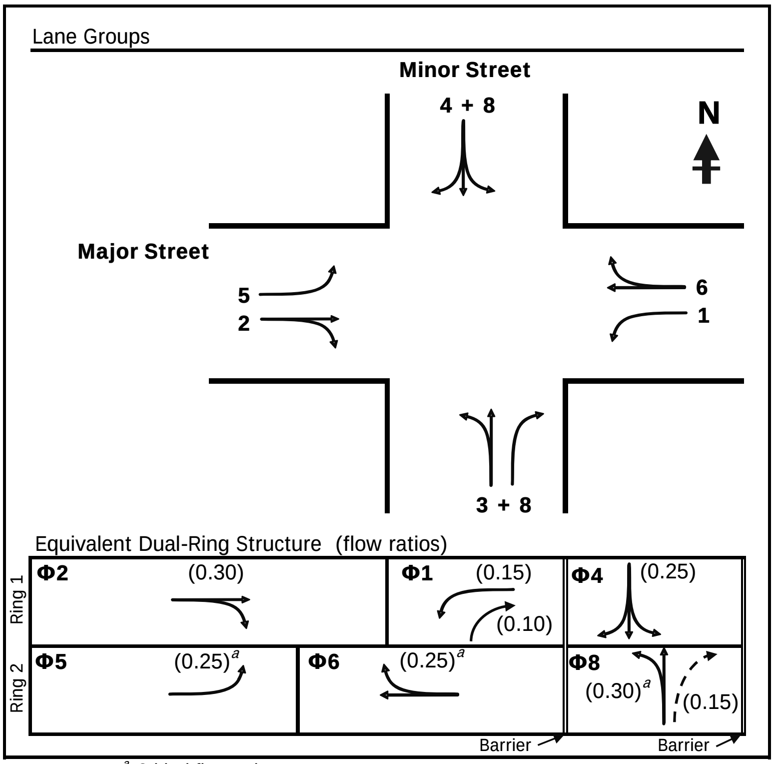
Finally, the process is repeated for the phases between the other barrier pair. One critical flow rate is defined for each barrier pair by this process. These two values are then added to obtain the sum of the critical flow ratios used in Equation 18-17. The lost time associated with each of the critical phases is added to yield the cycle lost time L.

The procedure for the basic intersection case is explained in the next few paragraphs by using an example intersection. A variation of this procedure applies when protected-permitted left-turn operation is used with pretimed control. This variation is described after the basic case is described.

Basic Case

Consider a pretimed intersection with a lead–lag phase sequence on the major street and a permitted-only sequence on the minor street, as shown in Exhibit 18-15. The northbound right turn is provided an exclusive lane and a green arrow indication that displays concurrently with the complementary left-turn phase on the major street. Each of the left-turn movements on the major street is served with a protected phase.

Exhibit 18-15
Critical Path Determination with Protected Left-Turn Phases



Note: [a] Critical flow ratio.

Phases 4 and 8 represent the only phases between the barrier pair serving the minor-street movements. Inspection of the flow ratios provided in the exhibit indicates that Phase 8 has two lane-group flow rates. The larger flow rate corresponds to the shared left-turn and through movement. Thus, the phase flow ratio for Phase 8 is 0.30. The phase flow ratio for Phase 4 is 0.25. Of the two phases, the largest phase flow ratio is that associated with Phase 8 (= 0.30), so it represents the critical phase for this barrier pair.

Phases 1, 2, 5, and 6 represent the phases between the other barrier pair. They serve the major-street approaches. A flow ratio is shown for the right-turn lane group in Phase 1 because the intersection has pretimed control. If the intersection was actuated, it is unlikely that the right-turn detection would be used to extend Phase 1, and the flow ratio for the right-turn lane group would not be considered in defining the phase flow ratio for Phase 1. Regardless, the phase flow ratio of Phase 1 is 0.15, on the basis of the left-turn lane group flow rate.

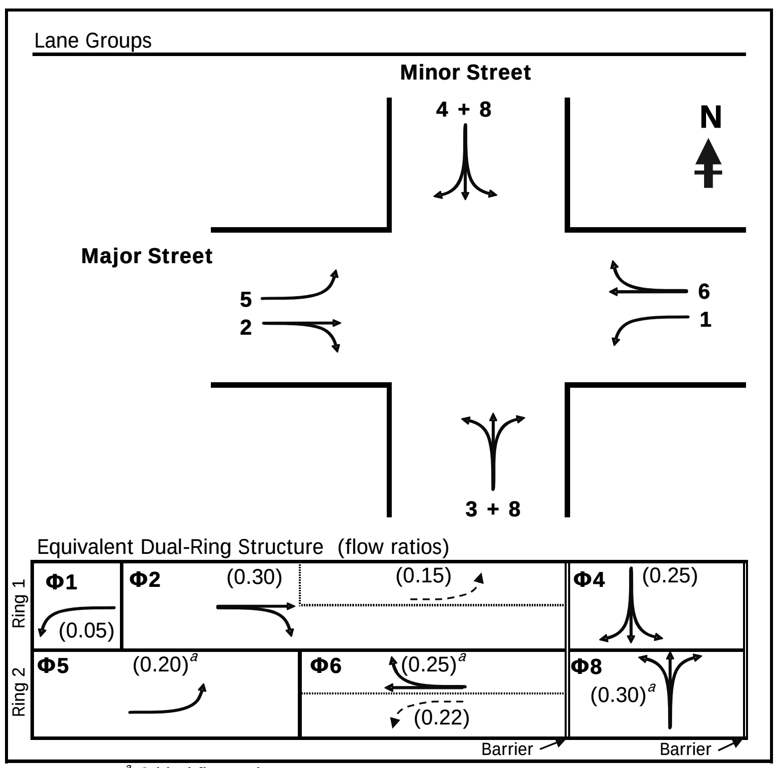
There are two possible critical paths through the major-street phase sequence—one path is associated with Phases 1 and 2 (i.e., Ring 1), and the other path is associated with Phases 5 and 6 (i.e., Ring 2). The total phase flow ratio for the Ring 1 path is 0.30 + 0.15, or 0.45. The total phase flow ratio for the Ring 2 path is 0.25 + 0.25 = 0.50. The latter total is larger and, hence, represents the critical path. It identifies Phases 5 and 6 as the critical phases. Thus, the sum of critical flow ratios for the cycle is 0.80 (= 0.30 + 0.50).

One increment of phase lost time l_t is associated with each phase on the critical path. Thus, the cycle lost time L is computed as the sum of the lost time for each of Phases 5, 6, and 8.

Special Case: Pretimed Protected-Permitted Left-Turn Operation

Consider a pretimed intersection with a lead–lead phase sequence on the major street and a permitted-only sequence on the minor street, as shown in Exhibit 18-16. The left-turn movements on the major street operate in the protected-permitted mode. Phases 4 and 8 represent the only phases between one barrier pair. They serve the minor-street lane groups. By inspection of the flow ratios provided in the exhibit, Phase 8 has the highest flow ratio (= 0.30) of the two phases and represents the critical phase for this barrier pair.

Exhibit 18-16
Critical Path Determination with Protected-Permitted Left-Turn Operation



Note: [a] Critical flow ratio.

Phases 1, 2, 5, and 6 represent the phases between the other barrier pair. They serve the major-street approaches. Each left-turn lane group is shown to be served during two phases—once during the left-turn phase and once during the phase serving the adjacent through movement. The flow ratio for each of the four left-turn service periods is shown in Exhibit 18-16. The following rules define the possible critical paths through this phase sequence:

1. One path is associated with Phases 1 and 2 in Ring 1 (0.35 = 0.05 + 0.30).
2. One path is associated with Phases 5 and 6 in Ring 2 (0.45 = 0.20 + 0.25).

3. If a lead–lead or lag–lag phase sequence is used, then one path is associated with (*a*) the left-turn phase with the larger flow ratio and (*b*) the through phase that permissively serves the same left-turn lane group. Sum the protected and permitted left-turn flow ratios on this path (0.35 = 0.20 + 0.15).
4. If a lead–lag phase sequence is used, then one path is associated with (*a*) the leading left-turn phase, (*b*) the lagging left-turn phase, and (*c*) the *controlling* through phase (see discussion to follow). Sum the two protected left-turn flow ratios and the one controlling permitted left-turn flow ratio on this path.

If a lead–lag phase sequence is used, each of the through phases that permissively serve a left-turn lane group is considered in determining the *controlling* through phase. If both through phases have a permitted period, then there are two through phases to consider. The controlling through phase is that phase with the larger permitted left-turn flow ratio. For example, if Phase 1 were shown to lag Phase 2 in Exhibit 18-16, then Phase 6 would be the controlling through phase because the permitted left-turn flow ratio of 0.22 exceeds 0.15. The critical path for this phase sequence would be 0.47 (= 0.20 + 0.22 + 0.05).

The first three rules in the preceding list apply to the example intersection. The calculations are shown for each path in parentheses in the previous list of rules. The total flow ratio for the path in Ring 2 is largest (= 0.45) and, hence, represents the critical path. It identifies Phases 5 and 6 as the critical phases. Thus, the sum of critical flow ratios for the cycle is 0.75 (= 0.30 + 0.45).

If Rule 3 in the preceding list applies, then the only lost time incurred is the start-up lost time l_1 associated with the first critical phase and the clearance lost time l_2 associated with the second critical phase. If Rule 1, 2, or 4 applies, then one increment of phase lost time l_t is associated with each critical phase. Rule 2 applies for the example, so the cycle lost time L is computed as the sum of the lost time for each of Phases 5, 6, and 8.

Two flow ratios are associated with Phase 6 in this example. Both flow ratios are shown possibly to dictate the duration of Phase 6 (this condition does not hold for Phase 2 because of the timing of the left-turn phases). This condition is similar to that for the northbound right-turn movement in Phase 1 of Exhibit 18-15 and the treatment is the same. That is, both flow ratios are considered in defining the phase flow ratio for Phase 6.

This example is specific to pretimed control. If actuated control were used, then it is unlikely that the left-turn detection on the major street would be used to extend the through phases. In this situation, the flow ratio for the permitted left-turn lane group would not be considered in defining the phase flow ratio for the through phases (i.e., only the first two rules in the previous list would apply). In short, the analysis of protected-permitted left-turn operation with actuated control defaults to the basic case previously described.

Step 8: Determine Delay

The delay calculated in this step represents the average control delay experienced by all vehicles that arrive during the analysis period. It includes any delay incurred by these vehicles that are still in queue after the analysis period ends. The control delay for a given lane group is computed by using Equation 18-19.

Equation 18-19

$$d = d_1 + d_2 + d_3$$

where

d = control delay (s/veh),

d_1 = uniform delay (s/veh),

d_2 = incremental delay (s/veh), and

d_3 = initial queue delay (s/veh).

Concepts

Uniform Delay

Equation 18-20 represents one way to compute delay when arrivals are assumed to be random throughout the cycle. It also assumes one effective green period during the cycle and one saturation flow rate during this period. It is based on the first term of a delay equation presented elsewhere (*6*).

Equation 18-20

$$d_1 = \frac{0.5\,C\,(1 - g/C)^2}{1 - \left[\min(1, X)g/C\right]}$$

All variables are as previously defined. The delay calculation procedure used in this methodology is consistent with Equation 18-20. However, it removes the aforementioned assumptions to allow more accurate uniform delay estimates for progressed traffic movements, movements with multiple green periods, and movements with multiple saturation flow rates (e.g., protected-permitted turn movements). It is called the "incremental queue accumulation" procedure (*21, 22*).

The incremental queue accumulation procedure models arrivals and departures as they occur during the average cycle. Specifically, it considers arrival rates and departure rates as they may occur during one or more effective green periods. The rates and resulting queue size can be shown in a queue accumulation polygon, such as that shown previously in Exhibit 18-14. The procedure decomposes the resulting polygon into an equivalent set of trapezoids or triangles for the purpose of delay estimation.

The key criterion for constructing a trapezoid or triangle is that the arrival and departure rates must be effectively constant during the associated time period. This process is illustrated in Exhibit 18-17 for a lane group having two different departure rates during the effective green period.

The delay associated with the cycle is determined by summing the area of the trapezoids or triangles that compose the polygon. The area of a given trapezoid or triangle is determined by first knowing the queue at the start of the interval and then adding the number of arrivals and subtracting the number of departures during the specified time interval. The result of this calculation yields the number of vehicles in queue at the end of the interval. Equation 18-21 illustrates this calculation for interval *i*.

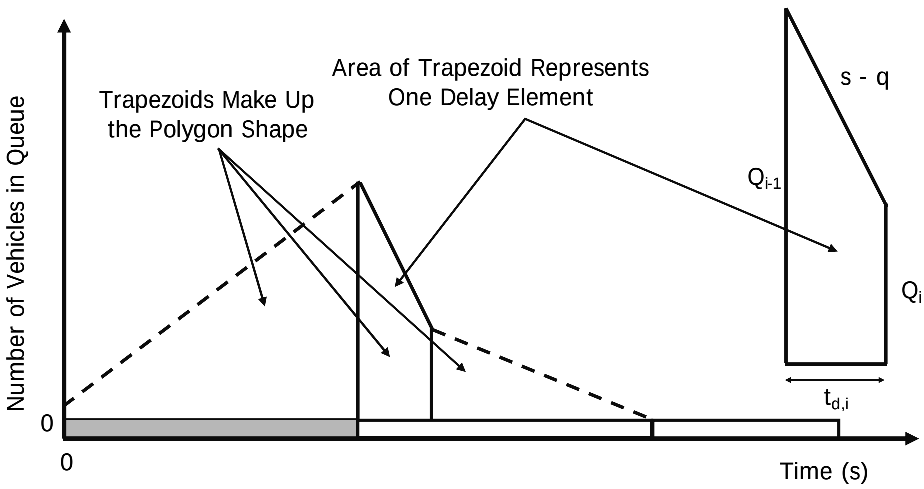


Exhibit 18-17
Decomposition of Queue Accumulation Polygon

Equation 18-21

$$Q_i = Q_{i-1} - (s/3{,}600 - q/N)\, t_{d,i} \quad \geq 0.0$$

where

Q_i = queue size at the end of interval *i* (veh),

q = arrival flow rate = v/3,600 (veh/s),

$t_{d,i}$ = duration of time interval *i* during which the arrival flow rate and saturation flow rate are constant (s), and

all other variables as previously defined.

Construction of the queue accumulation polygon requires converting all flow rate variables to common units of vehicles per second per lane. This conversion is implicit for all flow rate variables shown in exhibits here that depict a queue accumulation polygon.

Equation 18-22 is used to compute the *total* delay associated with a given trapezoid or triangle.

Equation 18-22

$$d_{T,i} = 0.5\,(Q_{i-1} + Q_i)\, t_{d,i}$$

where $d_{T,i}$ is the total delay associated with interval *i* (veh-s) and other variables are as previously defined. Total delay is computed for all intervals, added together, and the sum divided by the number of arrivals during the cycle (= qC) to estimate uniform delay in seconds per vehicle.

Construction of the queue accumulation polygon requires that the arrival flow rate not exceed the phase capacity. If the arrival flow rate exceeds capacity, then it is set to equal the capacity for the purpose of constructing the polygon. The queue can be assumed to equal zero at the end of the protected phase, and the polygon construction process begins at this point in the cycle. Once constructed, this assumption must be checked and, if the ending queue is not zero, then a second polygon is constructed with this ending queue as the starting queue for the first interval.

Polygon construction requires identifying points in the cycle where one of the following two conditions applies:

- The departure rate changes (e.g., due to the start or end of effective green, a change in the saturation flow rate, depletion of the subject queue, depletion of the opposing queue, sneakers depart).
- The arrival rate changes (e.g., when a platoon arrival condition changes).

During the intervals of time between these points, the saturation flow rate and arrival flow rate are constant.

The determination of flow-rate-change points may require an iterative calculation process when the approach has shared lanes. For example, an analysis of the opposing through movement must be completed to determine the time this movement's queue clears and the subject left-turn lane group can begin its service period. This service period may, in turn, dictate when the permitted left-turn movements on the opposing approach may depart.

The procedure is based on defining arrival rate as having one of two flow states: an arrival rate during the green indication and an arrival rate during the red indication. Further information about when each of these rates applies is described in the discussion for platoon ratio in the required input data subsection. The proportion of vehicles arriving during the green indication P is used to compute the arrival flow rate during each flow state. The following equations can be used to compute these rates:

Equation 18-23

$$q_g = \frac{q\,P}{g/C}$$

and

Equation 18-24

$$q_r = \frac{q\,(1-P)}{1-g/C}$$

where

q_g = arrival flow rate during the effective green time (veh/s),

q_r = arrival flow rate during the effective red time (veh/s), and

all other variables as previously defined.

A more detailed description of the procedure for constructing a queue accumulation polygon for lane groups with various lane allocations and operating modes is provided in Chapter 31.

Incremental Delay

Incremental delay consists of two delay components. One component accounts for delay due to the effect of random, cycle-by-cycle fluctuations in demand that occasionally exceed capacity. This delay is evidenced by the occasional overflow queue at the end of the green interval (i.e., cycle failure). The second component accounts for delay due to a sustained oversaturation during the analysis period. This delay occurs when aggregate demand during the analysis period exceeds aggregate capacity. It is sometimes referred to as the "deterministic" delay component and is shown as variable $d_{2,d}$ in Exhibit 18-18.

Exhibit 18-18
Cumulative Arrivals and Departures During an Oversaturated Analysis Period

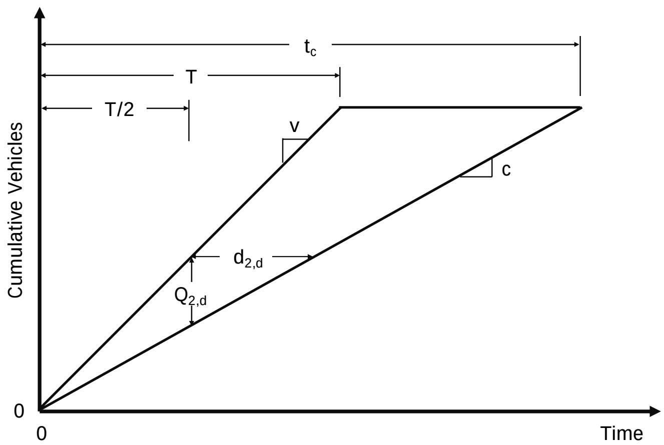


Exhibit 18-18 illustrates the queue growth that occurs as vehicles arrive at a demand flow rate v during analysis period T, which has capacity c. The deterministic delay component is represented by the triangular area bounded by the thick line and is associated with an average delay per vehicle represented by the variable $d_{2,d}$. The last vehicle to arrive during the analysis period is shown to clear the queue t_c hours after the start of the analysis period. The average queue size associated with this delay is also shown in the exhibit as $Q_{2,d}$. The queue present at the end of the analysis period [= $T\ (v - c)$] is referred to as the *residual queue.*

Initial Queue Delay

The equation used to estimate incremental delay is based on the assumption that no initial queue is present at the start of the analysis period. The initial queue delay term accounts for the additional delay incurred due to an initial queue. This queue is a result of unmet demand in the previous time period. It does *not* include any vehicles that may be in queue due to random, cycle-by-cycle fluctuations in demand that occasionally exceed capacity. When a multiple-period analysis is undertaken, the initial queue for the second and subsequent analysis periods is equal to the residual queue from the previous analysis period.

Exhibit 18-19 illustrates the delay due to an initial queue as a trapezoid shape bounded by thick lines. The average delay per vehicle is represented by the variable d_3. The initial queue size is shown as Q_b vehicles. The duration of time during the analysis period for which the effect of the initial queue is still present is represented by the variable t. This duration is shown to equal the analysis period in Exhibit 18-19. However, it can be less than the analysis period duration for some lower-volume conditions.

Exhibit 18-19 illustrates the case in which the demand flow rate v exceeds the capacity c during the analysis period. In contrast, Exhibit 18-20 and Exhibit 18-21 illustrate alternative cases in which the demand flow rate is less than the capacity.

Exhibit 18-19
Initial Queue Delay with Increasing Queue Size

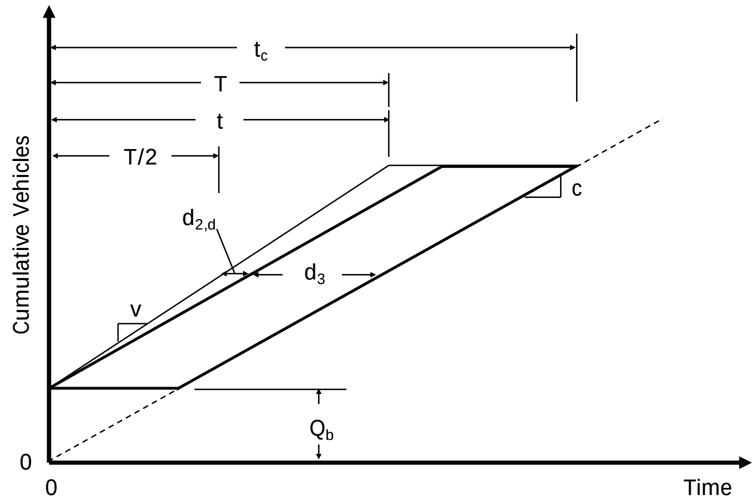


Exhibit 18-20
Initial Queue Delay with Decreasing Queue Size

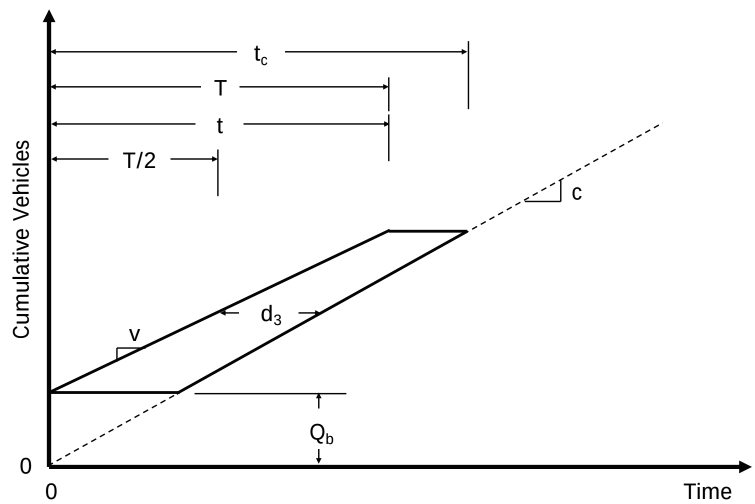
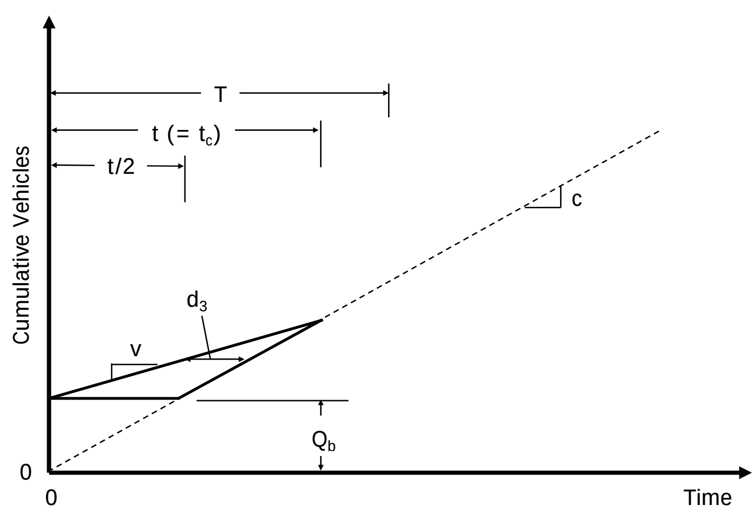


Exhibit 18-21
Initial Queue Delay with Queue Clearing



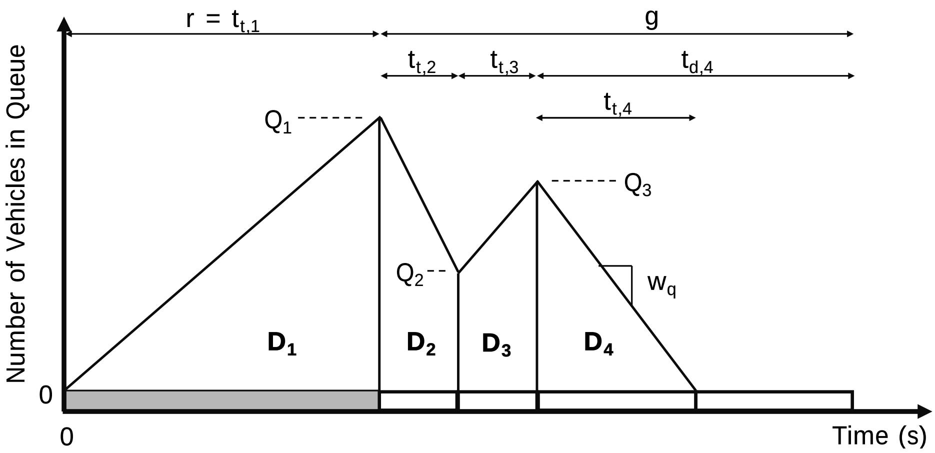
In this chapter, the phrase *initial queue* is always used in reference to the initial queue due to unmet demand in the previous time period. It *never* refers to vehicles in queue due to random, cycle-by-cycle fluctuations in demand.

The remainder of this step describes the procedure for computing the control delay for a lane group during a given analysis period. Chapter 31 describes a technique for measuring control delay in the field.

A. Compute Baseline Uniform Delay

Exhibit 18-14 was previously provided to illustrate a simple polygon for a lane group serving one traffic movement and for which there are no permitted or protected-permitted left-turn movements. Exhibit 18-22 is provided to illustrate delay calculation for a more complicated polygon shape. This particular polygon describes permitted left-turn operation from a shared lane for a specific combination of timing and volume conditions.

Exhibit 18-22
Polygon for Uniform Delay Calculation



The area bounded by the polygon represents the total delay incurred during the average cycle. The total delay is then divided by the number of arrivals per cycle to estimate the average uniform delay. These calculations are summarized in Equation 18-25, with Equation 18-26.

$$d_{1b} = \frac{0.5 \sum_{i=1} (Q_{i-1} + Q_i) t_{t,i}}{q C}$$

Equation 18-25

with

$$t_{t,i} = \min(t_{d,i}, Q_{i-1} / w_q)$$

Equation 18-26

where

d_{1b} = baseline uniform delay (s/veh),

$t_{t,i}$ = duration of trapezoid or triangle in interval *i* (s),

w_q = queue change rate (i.e., slope of the upper boundary of the trapezoid or triangle) (veh/s), and

all other variables as previously defined.

The summation term in Equation 18-25 includes all intervals for which there is a nonzero queue. In general, $t_{t,i}$ will equal the duration of the corresponding interval. However, during some intervals, the queue will dissipate and $t_{t,i}$ will only be as long as the time required for the queue to dissipate (= Q_{i-1}/w_q). This condition is shown to occur during Time Interval 4 in Exhibit 18-22.

The delay computed in this step is referred to as the *baseline uniform delay.* It may be adjusted in Step C if there is an initial queue that dissipates during the analysis period. The uniform delay to be used in Equation 18-19 is determined in this subsequent step.

B. Initial Queue Analysis

If an initial queue is present for any lane group at the intersection, then a second set of polygons needs to be constructed for each intersection lane group (in addition to those constructed for Step A). If no lane group has an initial queue, then this step is skipped.

At the start of this step, the initial queue that was input for each movement group needs to be converted to an initial queue for each lane group. When there is a one-to-one correlation between the movement group and the lane group, then the initial queue for the lane group equals the input initial queue for the movement group. When there is a shared lane on an approach that has another shared lane or additional through lanes, then the input initial queue needs to be distributed among the lane groups that serve the movements sharing the lane. Specifically, the initial queue for each lane group is estimated as being equal to the input initial queue multiplied by the number of lanes in the lane group and divided by the total number of shared and through lanes.

When the polygons are constructed in this step, lane groups with an initial queue will have their arrival flow rate set to equal the lane group capacity, regardless of their input arrival rate. The remaining lane groups will have their arrival flow rate set to equal the smaller of the input demand flow rate or the capacity. One polygon is constructed for each lane group, regardless of whether it has an initial queue.

The need for a second set of polygons stems from the influence one lane group often has on the operation of other lane groups. This influence is notably adverse when one or more lane groups are operating in a saturated state for a portion of the analysis period. If the saturated lane group represents a conflicting movement to a lane group that includes a permitted left-turn operation, then the left-turn lane group's operation will also be adversely affected for the same time period. Moreover, if the phase serving the lane group is actuated, then its capacity during the saturated state will be different from that of the subsequent unsaturated state. The following procedure is used to address this situation.

The duration of unmet demand is calculated in this step for each lane group with Equation 18-27 or Equation 18-28.

If $v \geq c_s$, then

Equation 18-27

$$t = T$$

If $v < c_s$, then

$$t = Q_b / (c_s - v) \quad \leq T$$

Equation 18-28

where

t = duration of unmet demand in the analysis period (h),

T = analysis period duration (h),

Q_b = initial queue at the start of the analysis period (veh),

v = demand flow rate (veh/h), and

c_s = saturated capacity (veh/h).

For this calculation, the saturated capacity c_s is equal to that obtained from the polygon constructed in this step and is reflective of the phase duration that is associated with saturated operation (due to the initial queue).

Next, the average duration of unmet demand is calculated with Equation 18-29.

$$t_a = \frac{1}{N_g} \sum_{i \in N_g} t_i$$

Equation 18-29

where

t_a = average duration of unmet demand in the analysis period (h), and

N_g = number of lane groups for which t exceeds 0.0 h.

The summation term in Equation 18-29 represents the sum of the t values for only those lane groups that have a value of t that exceeds 0.0 h. The average duration t_a is considered as a single representative value of t for all lane groups that do not have an initial queue.

The procedure described in Step A is repeated in this step to estimate the saturated uniform delay d_s for each lane group.

C. Compute Uniform Delay

If no lane group has an initial queue, then the uniform delay is equal to that computed in Step A (i.e., $d_1 = d_{1b}$). If an initial queue is present for any lane group at the intersection, then Equation 18-30 or Equation 18-31 is used to compute the uniform delay for each lane group.

If lane group i has an initial queue, then

$$d_{1,i} = d_{s,i} \frac{t_i}{T} + d_{1b,i} \frac{(T - t_i)}{T}$$

Equation 18-30

If lane group i does not have an initial queue, then

$$d_{1,i} = d_{s,i} \frac{t_a}{T} + d_{1b,i} \frac{(T - t_a)}{T}$$

Equation 18-31

where d_s is the saturated uniform delay (s/veh), t_i is the duration of unmet demand for lane group i in the analysis period (h), and other variables are as previously defined.

D. Compute Average Capacity

If no lane group has an initial queue, then the average lane group capacity c_A is equal to that computed in Step 7 (i.e., $c_A = c$). If an initial queue is present for any lane group at the intersection, then Equation 18-32 and Equation 18-33 are used to compute the average capacity for each lane group.

If lane group i has an initial queue, then

Equation 18-32

$$c_{A,i} = c_{s,i}\frac{t_i}{T} + c_i\frac{(T - t_i)}{T}$$

If lane group i does not have an initial queue, then

Equation 18-33

$$c_{A,i} = c_{s,i}\frac{t_a}{T} + c_i\frac{(T - t_a)}{T}$$

where c_A is the average capacity (veh/h) and other variables are as previously defined.

E. Compute Initial Queue Delay

If no lane group has an initial queue, then the initial queue delay d_3 is equal to 0.0 s/veh. If an initial queue is present for any lane group at the intersection, then Equation 18-34 through Equation 18-39 are used to compute the initial queue delay for each lane group.

Equation 18-34

$$d_3 = \frac{3{,}600}{vT}\left(t_A\frac{Q_b + Q_e - Q_{eo}}{2} + \frac{Q_e^2 - Q_{eo}^2}{2\,c_A} - \frac{Q_b^2}{2\,c_A}\right)$$

with

Equation 18-35

$$Q_e = Q_b + t_A(v - c_A)$$

If $v \geq c_A$, then

Equation 18-36

$$Q_{eo} = T(v - c_A)$$

Equation 18-37

$$t_A = T$$

If $v < c_A$, then

Equation 18-38

$$Q_{eo} = 0.0\ \text{veh}$$

Equation 18-39

$$t_A = Q_b/(c_A - v) \quad \leq T$$

where

t_A = adjusted duration of unmet demand in the analysis period (h),

Q_e = queue at the end of the analysis period (veh),

Q_{eo} = queue at the end of the analysis period when $v \geq c_A$ and $Q_b = 0.0$ (veh), and

other variables as previously defined.

The last vehicle that arrives to an overflow queue during the analysis period will clear the intersection at the time obtained with the following equation:

$$t_c = t_A + Q_e / c_A$$

Equation 18-40

where t_c is the queue clearing time (h) and other variables are as previously defined.

The queue clearing time is measured from the start of the analysis period to the time the last arriving vehicle clears the intersection.

F. Compute Incremental Delay Factor

The equation for computing incremental delay includes a variable that accounts for the effect of controller type on delay. This variable is referred to as the incremental delay factor *k*. It varies in value from 0.04 to 0.50. A factor value of 0.50 is recommended for pretimed phases, coordinated phases, and phases set to "recall-to-maximum."

An actuated phase has the ability to adapt its green interval duration to serve the demand on a cycle-by-cycle basis and, thereby, to minimize the frequency of cycle failure. Only when the green is extended to its maximum limit is this capability curtailed. This influence of actuated operation on delay is accounted for in Equation 18-41 through Equation 18-44.

$$k = (1 - 2k_{min})(v / c_a - 0.5) + k_{min} \quad \le 0.50$$

Equation 18-41

with

$$k_{min} = -0.375 + 0.354\,PT - 0.0910\,PT^2 + 0.00889\,PT^3 \quad \ge 0.04$$

Equation 18-42

$$c_a = 3{,}600\,\frac{g_a\, s\, N}{C}$$

Equation 18-43

$$g_a = G_{max} + Y + R_c - l_1 - l_2$$

Equation 18-44

where

k = incremental delay factor,

c_a = available capacity for a lane group served by an actuated phase (veh/h),

k_{min} = minimum incremental delay factor, and

g_a = available effective green time (s).

All other variables are as previously defined. As indicated by this series of equations, the factor value depends on the maximum green setting and the passage time setting for the phase that controls the subject lane group. Research indicates that shorter passage times result in a lower value of *k* (and lower delay), provided that the passage time is not so short that the phase terminates before the queue is served (*11*).

G. Compute Incremental Delay

The incremental delay term accounts for delay due to random variation in the number of arrivals on a cycle-by-cycle basis. It also accounts for delay caused by demand exceeding capacity during the analysis period. The amount by which demand exceeds capacity during the analysis period is referred to here as unmet demand. The incremental delay equation was derived by using an assumption of no initial queue due to unmet demand in the preceding analysis period. Equation 18-45, with Equation 18-46, is used to compute incremental delay.

Equation 18-45

$$d_2 = 900\,T\left[(X_A - 1) + \sqrt{(X_A - 1)^2 + \frac{8\,k\,I\,X_A}{c_A T}}\right]$$

with

Equation 18-46

$$X_A = v / c_A$$

where X_A is the average volume-to-capacity ratio and other variables are as previously defined. The incremental delay term is valid for all values of X_A, including highly oversaturated lane groups.

H. Compute Lane Group Control Delay

The uniform delay, incremental delay, and initial queue delay values computed in the previous steps are added (see Equation 18-19) to estimate the control delay for the subject lane group.

I. Compute Aggregated Delay Estimates

It is often desirable to compute the average control delay for the intersection approach. This aggregated delay represents a weighted average delay, where each lane group delay is weighted by the lane group demand flow rate. The approach control delay is computed with Equation 18-47.

Equation 18-47

$$d_{A,j} = \frac{\sum_{i=1}^{m_j} d_i v_i}{\sum_{i=1}^{m_j} v_i}$$

where

$d_{A,j}$ = approach control delay for approach j (s/veh),

d_i = control delay for lane group i (s/veh), and

m_j = number of lane groups on approach j.

All other variables are as previously defined. The summation terms in Equation 18-47 represent the sum over all lane groups on the subject approach.

Similarly, intersection control delay is computed with Equation 18-48.

Equation 18-48

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

where d_I is the intersection control delay (s/veh) and all other variables are as previously defined. The summation terms in Equation 18-48 represent the sum over all lane groups at the subject intersection.

Step 9. Determine LOS

Exhibit 18-4 is used to determine the LOS for each lane group, each approach, and the intersection as a whole. LOS is an indication of the acceptability of delay levels to motorists at the intersection. It can also indicate an unacceptable oversaturated operation for individual lane groups.

Step 10. Determine Queue Storage Ratio

A procedure is described in Chapter 31 for estimating the back-of-queue size and the queue storage ratio. The back of queue is the position of the vehicle stopped farthest from the stop line during the cycle as a consequence of the display of a red signal indication. The back-of-queue size depends on the arrival pattern of vehicles and on the number of vehicles that do not clear the intersection during the previous cycle.

The queue storage ratio represents the proportion of the available queue storage distance that is occupied at the point in the cycle when the back-of-queue position is reached. If this ratio exceeds 1.0, then the storage space will overflow and queued vehicles may block other vehicles from moving forward.

Extension to Multiple Time Periods

The 10-step sequence can be extended to analysis of consecutive time periods, each of duration T, and each having a fixed demand flow rate. The analysis is performed for each analysis period in sequence, as they occur in time. The initial queue Q_b for the second and subsequent periods is equal to the final queue Q_e from the previous period.

Typically, a multiple-time-period analysis would start with an undersaturated time period, desirably one when there is no initial queue for any intersection movement group. The demand flow rate for each period is a required input.

Interpretation of Results

The computations discussed in the previous steps result in the estimation of control delay and LOS for each lane group, for each approach, and for the intersection as a whole. They also produce a volume-to-capacity ratio for each lane group and a critical intersection volume-to-capacity ratio. This part provides some useful interpretations of these performance measures.

Level of Service

In general, LOS is an indication of the *general* acceptability of delay to drivers. In this regard, it should be remembered that what might be acceptable in a large city is not necessarily acceptable in a smaller city or rural area.

Intersection LOS must be interpreted with caution. It can suggest acceptable operation of the intersection when in reality certain lane groups (particularly those with lower volumes) are operating at an unacceptable LOS but are masked at the intersection level by the acceptable performance of higher-volume lane groups. The analyst should always verify that each lane group is providing acceptable operation and consider reporting the LOS for the poorest-performing lane group as a means of providing context to the interpretation of intersection LOS.

Volume-to-Capacity Ratio

In general, a volume-to-capacity ratio greater than 1.0 is an indication of actual or potential breakdown. In such cases, a multiple-period analysis is advised for this condition. This analysis would encompass all consecutive periods in which a residual queue is present.

The critical intersection volume-to-capacity ratio from Equation 18-17 is useful in evaluating the intersection from a capacity-only perspective. It is possible to have a critical intersection volume-to-capacity ratio of less than 1.0 and still have individual movements oversaturated within the signal cycle. If this situation occurs, then the cycle time is generally not appropriately allocated among the phases. Reallocation of the cycle time should be considered, where additional time is given to the phases serving those lane groups with a volume-to-capacity ratio greater than 1.0.

A critical intersection volume-to-capacity ratio greater than 1.0 indicates that the overall signal timing and geometric design provide inadequate capacity for the given demand flows. Improvements that might be considered include the following:

- Basic changes in intersection geometry (i.e., change in the number or use of lanes),
- Increase in signal cycle length if it is determined to be too short, and
- Changes in signal phase sequence or timing.

Local guidelines should always be consulted before potential improvements are developed.

Fully actuated control is intended to allocate cycle time dynamically to movements on the basis of demand and, thereby, maintain efficient operation on a cycle-by-cycle basis. The critical intersection volume-to-capacity ratio can provide an indication of this efficiency. In general, this ratio will vary between 0.85 and 0.95 for most actuated intersections, with lower values in this range more common for intersections having multiple detectors in the through traffic lanes. A ratio less than 0.85 may be an indication of excessive green extension by random arrivals, and the analyst may consider reducing passage time, minimum green, or both. A ratio that is more than 0.95 may be an indication of frequent phase termination by max out and limited ability of the controller to reallocate cycle time dynamically on the basis of detected demand. Increasing the maximum green may improve operation in some instances; however, it may also degrade operation when phase flow rates vary widely (because green extension is based on total flow rate served by the phase, not flow rate per lane).

For semiactuated and coordinated-actuated control, the critical intersection volume-to-capacity ratio can vary widely because of the nonactuated nature of some phases. The duration of these phases may not be directly related to their associated demand; instead, it may be dictated by coordination timing or the demand for the other phases. A critical intersection volume-to-capacity ratio that exceeds 0.95 has the same interpretation as offered previously for fully actuated control.

The critical intersection volume-to-capacity ratio can be misleading when it is used to evaluate the overall sufficiency of the intersection geometry, as is often required in planning applications. The problem is that low flow rates dictate the need for short cycle lengths to minimize delay. Yet, Equation 18-17 indicates that the desired shorter cycle length produces a higher volume-to-capacity ratio. Therefore, a relatively large value of X_c (provided that it is less than 1.0) is not a certain indication of poor operation. Rather, it means that closer attention must be paid to the adequacy of phase duration and queue size, especially for the critical phases.

Volume-to-Capacity Ratio and Delay Combinations

In some cases, delay is high even when the volume-to-capacity ratio is low. In these situations, poor progression, a notably long cycle length, or an inefficient phase plan is generally the cause. When the intersection is part of a coordinated system, the cycle length is determined by system considerations, and alterations at individual intersections may not be practical.

It is possible that delay is at acceptable levels even when the volume-to-capacity ratio is high. This situation can occur when some combination of the following conditions exists: the cycle length is relatively short, the analysis period is short, the lane group capacity is high, and there is no initial queue. If a residual queue is created in this scenario, then the conduct of a multiple-period analysis is necessary to gain a true picture of the delay.

When both delay levels and volume-to-capacity ratios are unacceptably high, the situation is critical. In such situations, the delay may increase rapidly with small changes in demand. The full range of potential geometric and signal design changes should be considered in the search for improvements.

In summary, unacceptable delay can exist when capacity is a problem as well as when capacity is adequate. Further, acceptable delay levels do not automatically ensure that capacity is sufficient. Delay and capacity are complex variables that are influenced by a wide range of traffic, roadway, and signalization conditions. The methodology presented here can be used to estimate these performance measures, identify possible problems, and assist in developing alternative improvements.

PEDESTRIAN MODE

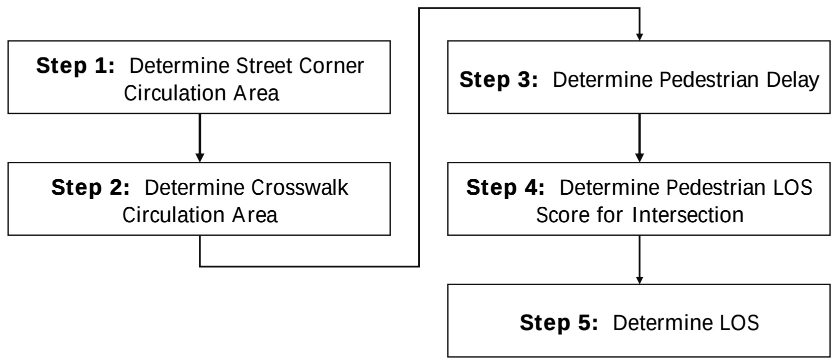
This subsection describes the methodology for evaluating the performance of a signalized intersection in terms of its service to pedestrians.

Intersection performance is separately evaluated for each crosswalk and intersection corner with this methodology. *Unless otherwise stated, all variables identified in this subsection are specific to one crosswalk and one corner.* A crosswalk is assumed to exist across each intersection leg unless crossing is specifically prohibited by local ordinance (and signed to this effect).

The methodology is focused on the analysis of signalized intersection performance. Chapter 17, Urban Street Segments, and Chapter 19, Two-Way STOP-Controlled Intersections, describe methodologies for evaluating the performance of these system elements with respect to the pedestrian mode.

The pedestrian methodology is applied through a series of five steps that determine the pedestrian LOS for a crosswalk and associated corners. These steps are illustrated in Exhibit 18-23.

Exhibit 18-23
Pedestrian Methodology for Signalized Intersections



Concepts

Performance Measures

The methodology provides a variety of measures for evaluating intersection performance in terms of its service to pedestrians. Each measure describes a different aspect of the pedestrian trip through the intersection. Performance measures that are estimated include the following:

- Corner circulation area,
- Crosswalk circulation area,
- Pedestrian delay, and
- Pedestrian LOS score.

The first two performance measures listed are based on the concept of "circulation area." One measure is used to evaluate the circulation area provided to pedestrians while they wait at the corner. Another measure is used to evaluate the area provided while the pedestrian is crossing in the crosswalk. Circulation area describes the space available to the average pedestrian. A larger area is more desirable from the pedestrian perspective. Exhibit 18-24 can be used to evaluate intersection performance from a circulation-area perspective.

Exhibit 18-24
Qualitative Description of Pedestrian Space

Pedestrian Space (ft^2/p)	Description
>60	Ability to move in desired path, no need to alter movements
>40–60	Occasional need to adjust path to avoid conflicts
>24–40	Frequent need to adjust path to avoid conflicts
>15–24	Speed and ability to pass slower pedestrians restricted
>8–15	Speed restricted, very limited ability to pass slower pedestrians
≤8	Speed severely restricted, frequent contact with other users

Pedestrian delay represents the average time a pedestrian waits for a legal opportunity to cross an intersection leg. The LOS score is an indication of the typical pedestrian's perception of the overall crossing experience.

Flow Conditions

Exhibit 18-25 and Exhibit 18-26 show the variables considered when one corner and its two crosswalks are evaluated. Two flow conditions are illustrated.

Condition 1 corresponds to the minor-street crossing that occurs during the major-street through phase. The pedestrians who desire to cross the major street must wait at the corner. Condition 2 corresponds to the major-street crossing that occurs during the minor-street through phase. For this condition, the pedestrians who desire to cross the minor street wait at the corner.

Exhibit 18-25
Condition 1: Minor-Street Crossing

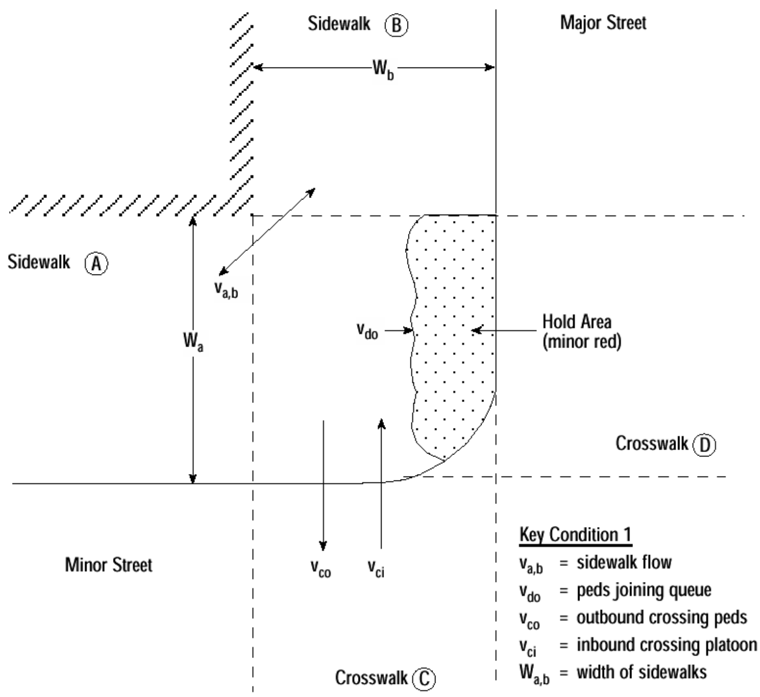
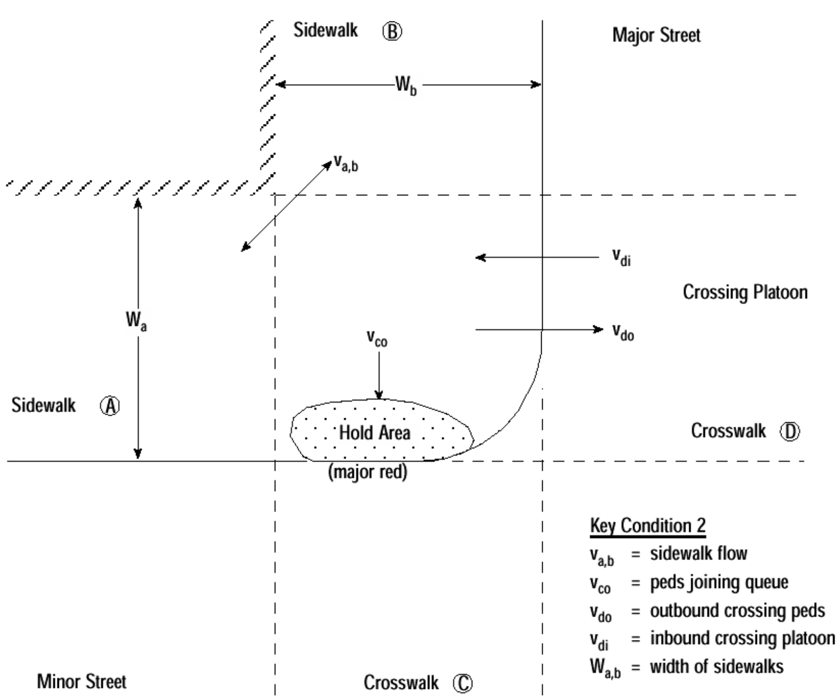


Exhibit 18-26
Condition 2: Major-Street Crossing



Effective Walk Time

Research indicates that, at intersections with pedestrian signal heads, pedestrians typically continue to enter the intersection during the first few seconds of the pedestrian clear interval (*26*, *28*). This behavior effectively increases the effective walk time available to pedestrians. A conservative estimate of this additional walk time is 4.0 s (*26*). A nonzero value for this additional time implies that some pedestrians are initiating their crossing during the flashing DON'T WALK indication.

The following guidance is provided to estimate the effective walk time on the basis of the aforementioned research findings. If the phase providing service to the pedestrians is either (*a*) actuated with a pedestrian signal head and rest-in-walk *not* enabled or (*b*) pretimed with a pedestrian signal head, then

Equation 18-49

$$g_{\text{Walk}} = Walk + 4.0$$

If the phase providing service to the pedestrians is actuated with a pedestrian signal head and rest-in-walk enabled, then

Equation 18-50

$$g_{\text{Walk}} = D_p - Y - R_c - PC + 4.0$$

Otherwise (i.e., no pedestrian signal head)

Equation 18-51

$$g_{\text{Walk}} = D_p - Y - R_c$$

where

g_{Walk} = effective walk time (s),

$Walk$ = pedestrian walk setting (s),

PC = pedestrian clear setting (s),

D_p = phase duration (s),

Y = yellow change interval (s), and

R_c = red clearance interval (s).

The aforementioned research indicates that the effective walk time estimated with Equation 18-49 or Equation 18-50 can vary widely among intersections. At a given intersection, the additional walk time can vary from 0.0 s to an amount equal to the pedestrian clear interval. The amount of additional walk time used by pedestrians depends on many factors, including the extent of pedestrian delay, vehicular volume, level of enforcement, and presence of countdown pedestrian signal heads.

The effective walk time estimated with Equation 18-49 or Equation 18-50 is considered to be directly applicable to design or planning analyses because it is conservative in the amount of additional walk time that it includes. A larger value of effective walk time may be applicable to an operational analysis if (*a*) field observation or experience indicates such a value would be consistent with actual pedestrian use of the flashing DON'T WALK indication; (*b*) an accurate estimate of pedestrian delay or queue size is desired; and (*c*) the predicted performance estimates are understood to reflect some illegal pedestrian behavior, possibly in response to constrained spaces or inadequate signal timing.

Step 1: Determine Street Corner Circulation Area

This step describes a procedure for evaluating the performance of one intersection corner. It is repeated for each intersection corner of interest.

The analysis of circulation area at the street corners and in the crosswalks compares available time and space with pedestrian demand. The product of time and space is the critical parameter. It combines the constraints of physical design (which limits available space) and signal operation (which limits available time). This parameter is hereafter referred to as "time–space."

A. Compute Available Time–Space

The total time–space available for circulation and queuing in the intersection corner equals the product of the net corner area and the cycle length C. Equation 18-52 is used to compute time–space available at an intersection corner. Exhibit 18-10 identifies the variables used in the equation.

Equation 18-52

$$TS_{corner} = C\,(W_a W_b - 0.215\,R^2)$$

where

TS_{corner} = available corner time–space (ft²-s),

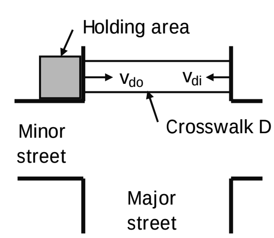
C = cycle length (s),

W_a = total walkway width of Sidewalk A (ft),

W_b = total walkway width of Sidewalk B (ft), and

R = radius of corner curb (ft).

If the corner curb radius is larger than either W_a or W_b, then the variable R in Equation 18-52 should equal the smaller of W_a or W_b.



B. Compute Holding-Area Waiting Time

The average pedestrian holding time represents the average time that pedestrians wait to cross the street when departing from the subject corner. The equation for computing this time is based on the assumption that pedestrian arrivals are uniformly distributed during the cycle. For Condition 1, as shown in Exhibit 18-25, Equation 18-53 and Equation 18-54 are used to compute holding-area time for pedestrians waiting to cross the major street.

Equation 18-53

$$Q_{tdo} = \frac{N_{do}(C - g_{\text{Walk},mi})^2}{2C}$$

with

Equation 18-54

$$N_{do} = \frac{v_{do}}{3{,}600}\,C$$

where

Q_{tdo} = total time spent by pedestrians waiting to cross the major street during one cycle (p-s),

N_{do} = number of pedestrians arriving at the corner each cycle to cross the major street (p),

$g_{\text{Walk},mi}$ = effective walk time for the phase serving the minor-street through movement (s),

C = cycle length (s), and

v_{do} = flow rate of pedestrians arriving at the corner to cross the major street (p/h).

If the phase providing service to the pedestrians is either (*a*) actuated with a pedestrian signal head and rest-in-walk *not* enabled or (*b*) pretimed with a pedestrian signal head, then

Equation 18-55

$$g_{\text{Walk},mi} = Walk_{mi} + 4.0$$

If the phase providing service to the pedestrians is actuated with a pedestrian signal head and rest-in-walk enabled, then

Equation 18-56

$$g_{\text{Walk},mi} = D_{p,mi} - Y_{mi} - R_{c,mi} - PC_{mi} + 4.0$$

Otherwise (i.e., no pedestrian signal head)

Equation 18-57

$$g_{\text{Walk},mi} = D_{p,mi} - Y_{mi} - R_{c,mi}$$

where

$g_{\text{Walk},mi}$ = effective walk time for the phase serving the minor-street through movement (s),

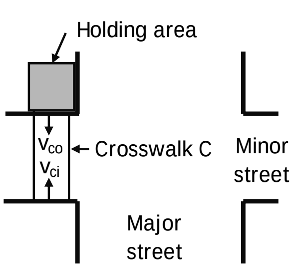
$Walk_{mi}$ = pedestrian walk setting for the phase serving the minor-street through movement (s),

PC_{mi} = pedestrian clear setting for the phase serving the minor-street through movement (s),

$D_{p,mi}$ = duration of the phase serving the minor-street through movement (s),

Y_{mi} = yellow change interval of the phase serving the minor-street through movement (s), and

$R_{c,mi}$ = red clearance interval of the phase serving the minor-street through movement (s).



For Condition 2, the previous three equations are repeated to compute the holding-area time for pedestrians waiting to cross the minor street Q_{tco}. For this application, the subscript letters "*do*" are replaced with the letters "*co*" to denote the pedestrians arriving at the corner to cross in Crosswalk C. Similarly, the subscript letters "*mi*" are replaced with "*mj*" to denote signal timing variables associated with the phase serving the major-street through movement.

C. Compute Circulation Time–Space

The time–space available for circulating pedestrians equals the total available time–space minus the time–space occupied by the pedestrians waiting to cross. The latter value equals the product of the total waiting time and the area used by waiting pedestrians (= 5.0 ft²/p). Equation 18-58 is used to compute the time–space available for circulating pedestrians.

Equation 18-58

$$TS_c = TS_{corner} - \left[5.0\,(Q_{tdo} + Q_{tco})\right]$$

where TS_c is the time–space available for circulating pedestrians (ft^2-s) and other variables are as previously defined.

D. Compute Pedestrian Corner Circulation Area

The space required for circulating pedestrians is computed by dividing the time–space available for circulating pedestrians by the time that pedestrians consume walking through the corner area. The latter quantity equals the total circulation volume multiplied by the assumed average circulation time (= 4.0 s). Equation 18-59, with Equation 18-60, is used to compute corner circulation area.

Equation 18-59

$$M_{\text{corner}} = \frac{TS_c}{4.0\,N_{tot}}$$

with

Equation 18-60

$$N_{tot} = \frac{v_{ci} + v_{co} + v_{di} + v_{do} + v_{a,b}}{3{,}600}\,C$$

where

M_{corner} = corner circulation area per pedestrian (ft^2/p),

N_{tot} = total number of circulating pedestrians that arrive each cycle (p),

v_{ci} = flow rate of pedestrians arriving at the corner after crossing the minor street (p/h),

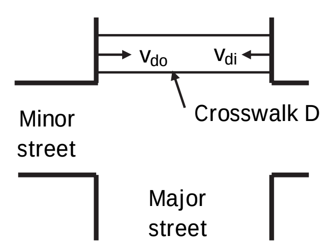
v_{co} = flow rate of pedestrians arriving at the corner to cross the minor street (p/h),

v_{di} = flow rate of pedestrians arriving at the corner after crossing the major street (p/h), and

$v_{a,b}$ = flow rate of pedestrians traveling through the corner from Sidewalk A to Sidewalk B, or vice versa (p/h).

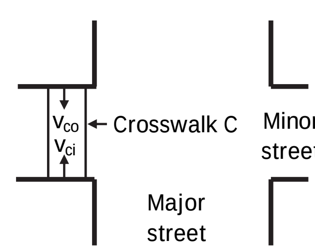
Other variables are as previously defined. The circulation area obtained from Equation 18-59 can be compared with the ranges provided in Exhibit 18-24 to make some judgments about the performance of the subject intersection corner.

Step 2: Determine Crosswalk Circulation Area



This step describes a procedure for evaluating the performance of one crosswalk. It is repeated for each crosswalk of interest.

The procedure to follow describes the evaluation of Crosswalk D in Exhibit 18-26 (i.e., a crosswalk across the major street). The procedure is repeated to evaluate Crosswalk C in Exhibit 18-25. For the second application, the subscript letters "*do*" and "*di*" are replaced with the letters "*co*" and "*ci*," respectively, to denote the pedestrians associated with Crosswalk C. Similarly, the subscript letter "*d*" is replaced with the letter "*c*" to denote the length and width of Crosswalk C. Also, the subscript letters "*mi*" are replaced with "*mj*" to denote signal timing variables associated with the phase serving the major-street through movement.



A. Establish Walking Speed

The recommended walking speeds reflect average (50th percentile) walking speeds for the purposes of calculating LOS. Traffic signal timing for pedestrians is typically based on a 15th percentile walking speed.

The average pedestrian walking speed S_p is needed to evaluate corner and crosswalk performance. Research indicates that the walking speed is influenced by pedestrian age and sidewalk grade (*26*). If 0% to 20% of pedestrians traveling along the subject segment are elderly (i.e., 65 years of age or older), an average walking speed of 4.0 ft/s is recommended for intersection evaluation. If more than 20% of all pedestrians are elderly, an average walking speed of 3.3 ft/s is recommended. In addition, an upgrade of 10% or greater reduces walking speed by 0.3 ft/s.

B. Compute Available Time–Space

Equation 18-61 is used to compute the time–space available in the crosswalk.

Equation 18-61

$$TS_{cw} = L_d W_d \, g_{\mathrm{Walk},mi}$$

where

TS_{cw} = available crosswalk time–space (ft²-s),

L_d = length of Crosswalk D (ft),

W_d = effective width of Crosswalk D (ft), and

$g_{\mathrm{Walk},mi}$ = effective walk time for the phase serving the minor-street through movement (s).

C. Compute Effective Available Time–Space

The available crosswalk time–space is adjusted in this step to account for the effect turning vehicles have on pedestrians. This adjustment is based on the assumed occupancy of a vehicle in the crosswalk. The vehicle occupancy is computed as the product of vehicle swept-path, crosswalk width, and the time the vehicle preempts this space. Equation 18-62 through Equation 18-64 are used for this purpose.

Equation 18-62

$$TS^*_{cw} = TS_{cw} - TS_{tv}$$

with

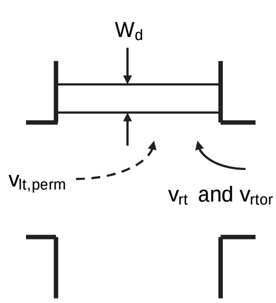
Equation 18-63

$$TS_{tv} = 40 \, N_{tv} W_d$$

Equation 18-64

$$N_{tv} = \frac{v_{lt,perm} + v_{rt} - v_{rtor}}{3{,}600} C$$

where



TS_{cw}^* = effective available crosswalk time–space (ft²-s),

TS_{tv} = time–space occupied by turning vehicles (ft²-s),

N_{tv} = number of turning vehicles during the walk and pedestrian clear intervals (veh),

$v_{lt,perm}$ = permitted left-turn demand flow rate (veh/h),

v_{rt} = right-turn demand flow rate (veh/h), and

v_{rtor} = right-turn-on-red flow rate (veh/h).

Other variables are as previously defined. The constant 40 in Equation 18-63 represents the product of the swept-path for most vehicles (= 8 ft) and the time that a turning vehicle occupies the crosswalk (= 5 s). The left-turn and right-turn flow rates used in Equation 18-64 are those associated with movements that receive a green indication concurrently with the subject pedestrian crossing *and* turn across the subject crosswalk.

D. Compute Pedestrian Service Time

Total service time is computed with either Equation 18-65 or Equation 18-66, depending on the crosswalk width, along with Equation 18-67. This time represents the elapsed time starting with the first pedestrian's departure from the corner to the last pedestrian's arrival at the far side of the crosswalk. In this manner, it accounts for platoon size in the service time (*29*).

If crosswalk width W_d is greater than 10 ft, then

$$t_{ps,do} = 3.2 + \frac{L_d}{S_p} + 2.7 \frac{N_{ped,do}}{W_d}$$

Equation 18-65

If crosswalk width W_d is less than or equal to 10 ft, then

$$t_{ps,do} = 3.2 + \frac{L_d}{S_p} + 0.27\ N_{ped,do}$$

Equation 18-66

with

$$N_{ped,do} = N_{do} \frac{C - g_{\text{Walk},mi}}{C}$$

Equation 18-67

where

$t_{ps,\,do}$ = service time for pedestrians that arrive at the corner to cross the major street (s),

$N_{ped,do}$ = number of pedestrians waiting at the corner to cross the major street (p), and

other variables are as previously defined.

Equation 18-67 provides an estimate of the number of pedestrians who cross as a group following the presentation of the WALK indication (or green indication, if pedestrian signal heads are not provided). It is also used to compute $N_{ped,di}$ for the other travel direction in the same crosswalk (using N_{di}, as defined below). Finally, Equation 18-65 or Equation 18-66 is used to compute the service time for pedestrians who arrive at the subject corner having waited on the other corner before crossing the major street $t_{ps,\,di}$ (using $N_{ped,di}$).

E. Compute Crosswalk Occupancy Time

The total crosswalk occupancy time is computed as a product of the pedestrian service time and the number of pedestrians using the crosswalk during one signal cycle. Equation 18-68 is used, with Equation 18-69 and results from previous steps, for the computation.

Equation 18-68

$$T_{occ} = t_{ps,do} N_{do} + t_{ps,di} N_{di}$$

with

Equation 18-69

$$N_{di} = \frac{v_{di}}{3{,}600} C$$

where

T_{occ} = crosswalk occupancy time (p-s), and

N_{di} = number of pedestrians arriving at the corner each cycle having crossed the major street (p).

Other variables are as previously defined.

F. Compute Pedestrian Crosswalk Circulation Area

The circulation space provided for each pedestrian is determined by dividing the time–space available for crossing by the total occupancy time, as shown in Equation 18-70.

Equation 18-70

$$M_{cw} = \frac{TS^{*}_{cw}}{T_{occ}}$$

where M_{cw} is the crosswalk circulation area per pedestrian (ft^2/p) and other variables are as previously defined.

The circulation area obtained from Equation 18-70 can be compared with the ranges provided in Exhibit 18-24 to make some judgments about the performance of the subject-intersection crosswalk (for the specified direction of travel). For a complete picture of the subject crosswalk's performance, the procedure described in this step should be repeated for the other direction of travel along the crosswalk (i.e., by using the other corner associated with the crosswalk as the point of reference).

Step 3: Determine Pedestrian Delay

This step describes a procedure for evaluating the performance of a crosswalk at the intersection. It is repeated for each crosswalk of interest.

The discussion that follows describes the evaluation of Crosswalk D shown in Exhibit 18-26. The procedure is applied again to evaluate Crosswalk C shown in Exhibit 18-25. For the second application, the subscript letters "*mi*" are replaced with "*mj*" to denote signal timing variables associated with the phase serving the major-street through movement.

The pedestrian delay while waiting to cross the major street is computed with Equation 18-71.

Equation 18-71

$$d_p = \frac{(C - g_{\text{Walk},mi})^2}{2C}$$

where d_p is pedestrian delay (s/p) and other variables are as previously defined.

The delay obtained from Equation 18-71 applies equally to both directions of travel along the crosswalk.

Research indicates that average pedestrian delay at signalized intersection crossings is not constrained by capacity, even when pedestrian flow rates reach 5,000 p/h (*26*). For this reason, delay due to oversaturated conditions is not included in the value obtained from Equation 18-71.

If the subject crosswalk is closed, then the pedestrian delay d_p is estimated as the value obtained from Equation 18-71 for the subject crosswalk, plus two increments of the delay from this equation when applied to the perpendicular crosswalk. This adjustment reflects the additional delay pedestrians incur when crossing the other three legs of the intersection so that they can continue walking in the desired direction.

The pedestrian delay computed in this step can be used to make some judgment about pedestrian compliance. In general, pedestrians become impatient when they experience delays in excess of 30 s/p, and there is a high likelihood of their not complying with the signal indication (*30*). In contrast, pedestrians are very likely to comply with the signal indication if their expected delay is less than 10 s/p.

Step 4: Determine Pedestrian LOS Score for Intersection

This step describes a procedure for evaluating the performance of one crosswalk. It is repeated for each crosswalk of interest.

The procedure to follow describes the evaluation of Crosswalk D in Exhibit 18-26. The procedure is repeated to evaluate Crosswalk C in Exhibit 18-25. For the second application, the subscript letter "*d*" is replaced with the letter "*c*" to denote the length and width of Crosswalk C. Also, the subscript letters "*mj*" are replaced with "*mi*" to denote variables associated with the minor street.

The pedestrian LOS score for the intersection $I_{p,int}$ is calculated by using Equation 18-72 through Equation 18-77.

$$I_{p,int} = 0.5997 + F_w + F_v + F_S + F_{\text{delay}}$$

Equation 18-72

with

$$F_w = 0.681 \left(N_d\right)^{0.514}$$

Equation 18-73

$$F_v = 0.00569 \left(\frac{v_{rtor} + v_{lt,perm}}{4}\right) - N_{rtci,d}\left(0.0027\ n_{15,mj} - 0.1946\right)$$

Equation 18-74

$$F_S = 0.00013\ n_{15,mj}\ S_{85,mj}$$

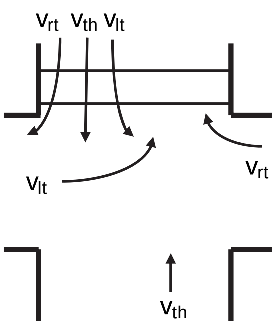
Equation 18-75

$$F_{\text{delay}} = 0.0401\ \ln(d_{p,d})$$

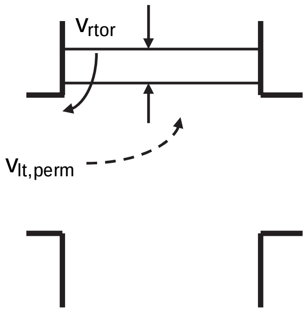
Equation 18-76

$$n_{15,mj} = \frac{0.25}{N_d} \sum_{i \in m_d} v_i$$

Equation 18-77



Movements crossing Crosswalk D



where

$I_{p,int}$ = pedestrian LOS score for intersection,

F_w = cross-section adjustment factor,

F_v = motorized vehicle volume adjustment factor,

F_S = motorized vehicle speed adjustment factor,

F_{delay} = pedestrian delay adjustment factor,

$\ln(x)$ = natural logarithm of x,

N_d = number of traffic lanes crossed when traversing Crosswalk D (ln),

$N_{rtci,d}$ = number of right-turn channelizing islands along Crosswalk D,

$n_{15,mj}$ = count of vehicles traveling on the major street during a 15-min period (veh/ln),

$S_{85,mj}$ = 85th percentile speed at a midsegment location on the major street (mi/h),

$d_{p,d}$ = pedestrian delay when traversing Crosswalk D (s/p),

v_i = demand flow rate for movement *i* (veh/h), and

m_d = set of all automobile movements that cross Crosswalk D (see figure in margin).

The left-turn flow rate $v_{lt,perm}$ used in Equation 18-74 is that associated with the left-turn movement that receives a green indication concurrently with the subject pedestrian crossing *and* turns across the subject crosswalk. The RTOR flow rate v_{rtor} is that associated with the approach being crossed and that turns across the subject crosswalk. It is not the same v_{rtor} used in Equation 18-64.

The pedestrian LOS score obtained from this equation applies equally to both directions of travel along the crosswalk.

The variable for "number of right-turn channelizing islands" N_{rtci} is an integer with a value of 0, 1, or 2.

Step 5: Determine LOS

This step describes a process for determining the LOS of one crosswalk. It is repeated for each crosswalk of interest.

The pedestrian LOS is determined by using the pedestrian LOS score from Step 4. This performance measure is compared with the thresholds in Exhibit 18-5 to determine the LOS for the subject crosswalk.

BICYCLE MODE

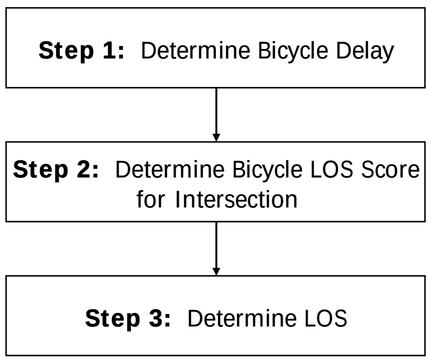
This subsection describes the methodology for evaluating the performance of a signalized intersection in terms of its service to bicyclists.

Intersection performance is evaluated separately for each intersection approach. *Unless otherwise stated, all variables identified in this subsection are specific to one intersection approach.* The bicycle is assumed to travel in the street (possibly in a bicycle lane) and in the same direction as adjacent motorized vehicles.

The methodology is focused on analyzing signalized intersection performance from the bicyclist point of view. Chapter 17, Urban Street Segments, describes a methodology for evaluating urban street performance.

The bicycle methodology is applied through a series of three steps that determine the bicycle LOS for an intersection approach. These steps are illustrated in Exhibit 18-27. Performance measures that are estimated include bicycle delay and a bicycle LOS score.

Exhibit 18-27
Bicycle Methodology for Signalized Intersections



Step 1: Determine Bicycle Delay

This step describes a procedure for evaluating the performance of one intersection approach. It is repeated for each approach of interest. Bicycle delay can be calculated only for intersection approaches that have an on-street bicycle lane or a shoulder that can be used by bicyclists as a bicycle lane. Bicyclists who share a lane with automobile traffic will incur the same delay as the automobiles.

A. Compute Bicycle Lane Capacity

A wide range of capacities and saturation flow rates have been reported by many countries for bicycle lanes at intersections. Research indicates that the base saturation flow rate may be as high as 2,600 bicycles/h (*31*). However, few intersections provide base conditions for bicyclists, and current information is insufficient to calibrate a series of appropriate saturation flow adjustment factors. Until such factors are developed, it is recommended that a saturation flow rate of 2,000 bicycles/h be used as an average value achievable at most intersections.

A saturation flow rate of 2,000 bicycles/h assumes that right-turning motor vehicles yield the right-of-way to through bicyclists. Where aggressive right-turning traffic exists, 2,000 bicycles/h may not be achievable. Local observations to determine a saturation flow rate are recommended in such cases.

The capacity of the bicycle lane at a signalized intersection may be computed with Equation 18-78.

Equation 18-78

$$c_b = s_b \frac{g_b}{C}$$

where

c_b = capacity of the bicycle lane (bicycles/h),

s_b = saturation flow rate of the bicycle lane = 2,000 (bicycles/h),

g_b = effective green time for the bicycle lane (s), and

C = cycle length (s).

The effective green time for the bicycle lane can be assumed to equal that for the adjacent motor-vehicle traffic stream that is served concurrently with the subject bicycle lane (i.e., $g_b = D_p - l_1 - l_2$).

B. Compute Bicycle Delay

Bicycle delay is computed with Equation 18-79.

Equation 18-79

$$d_b = \frac{0.5\,C\,(1 - g_b/C)^2}{1 - \min\left[\dfrac{v_{bic}}{c_b}, 1.0\right]\dfrac{g_b}{C}}$$

where d_b is bicycle delay (s/bicycle), v_{bic} is bicycle flow rate (bicycles/h), and other variables are as previously defined.

This delay equation is based on the assumption that there is no bicycle incremental delay or initial queue delay. Bicyclists will not normally tolerate an oversaturated condition and will select other routes or ignore traffic regulations to avoid the associated delays.

At most signalized intersections, the only delay to through bicycles is caused by the signal, because bicycles have the right-of-way over right-turning vehicles during the green indication. Bicycle delay could be longer than that obtained from Equation 18-79 when (*a*) bicycles are forced to weave with right-turning traffic during the green indication, or (*b*) drivers do not acknowledge the bicycle right-of-way because of high flows of right-turning vehicles.

The delay obtained from Equation 18-79 can be used to make some judgment about intersection performance. Bicyclists tend to have about the same tolerance for delay as pedestrians. They tend to become impatient when they experience a delay in excess of 30 s/bicycle. In contrast, they are very likely to comply with the signal indication if their expected delay is less than 10 s/bicycle.

Step 2: Determine Bicycle LOS Score for Intersection

This step describes a procedure for evaluating the performance of one intersection approach. It is repeated for each approach of interest. The bicycle LOS score can be calculated for any intersection approach, regardless of whether it has an on-street bicycle lane.

The bicycle LOS score for the intersection $I_{b,int}$ is calculated by using Equation 18-80 through Equation 18-83.

Equation 18-80

$$I_{b,int} = 4.1324 + F_w + F_v$$

with

Equation 18-81

$$F_w = 0.0153\,W_{cd} - 0.2144\,W_t$$

Equation 18-82

$$F_v = 0.0066\,\frac{v_{lt} + v_{th} + v_{rt}}{4\,N_{th}}$$

$$W_t = W_{ol} + W_{bl} + I_{pk} W_{os}^*$$

Equation 18-83

where

$I_{b,int}$ = bicycle LOS score for intersection;

W_{cd} = curb-to-curb width of the cross street (ft);

W_t = total width of the outside through lane, bicycle lane, and paved shoulder (ft);

v_{lt} = left-turn demand flow rate (veh/h);

v_{th} = through demand flow rate (veh/h);

v_{rt} = right-turn demand flow rate (veh/h);

N_{th} = number of through lanes (shared or exclusive) (ln);

W_{ol} = width of the outside through lane (ft);

W_{bl} = width of the bicycle lane = 0.0 if bicycle lane not provided (ft);

I_{pk} = indicator variable for on-street parking occupancy = 0 if $p_{pk} > 0.0$, 1 otherwise;

p_{pk} = proportion of on-street parking occupied (decimal);

W_{os} = width of paved outside shoulder (ft); and

W_{os}^* = adjusted width of paved outside shoulder; if curb is present $W_{os}^* = W_{os} - 1.5 \geq 0.0$, otherwise $W_{os}^* = W_{os}$ (ft).

The variable "proportion of on-street parking occupied" is used to describe the presence of on-street parking and activity on the approach and departure legs of the intersection that are used by the subject bicycle movement.

Step 3: Determine LOS

This step describes a process for determining the LOS of one intersection approach. It is repeated for each approach of interest.

The bicycle LOS is determined by using the bicycle LOS score from Step 2. This performance measure is compared with the thresholds in Exhibit 18-5 to determine the LOS for the subject approach.

3. APPLICATIONS

DEFAULT VALUES

Agencies that use the methodologies in this chapter are encouraged to develop a set of local default values based on field measurements at intersections in their jurisdiction. Local default values provide the best means of ensuring accuracy in the analysis results. In the absence of local default values, the values identified in this subsection can be used if the analyst believes they are reasonable for the intersection to which they are applied.

Exhibit 18-6, Exhibit 18-7, and Exhibit 18-9 identify the input data variables associated with the automobile, pedestrian, and bicycle methodologies. These variables can be categorized as (*a*) suitable for specification as a default value or (*b*) required input data. Those variables categorized as "suitable for specification as a default value" have a minor effect on performance estimates and tend to have a relatively narrow range of typical values used in practice. In contrast, required input variables have either a notable effect on performance estimates or a wide range of possible values.

Required input variables typically represent fundamental intersection geometric elements and demand flow rates. Values for these variables should be field-measured when possible.

If field measurement of the input variables is not possible, then various options exist for determining an appropriate value for a required input variable. As a first choice, input values should be established through the use of local guidelines. If local guidelines do not address the desired variable, then some input values may be determined by considering the typical operation of (or conditions at) similar intersections in the jurisdiction. As a last option, various authoritative national guideline documents are available and should be used to make informed decisions about design options and volume estimates. The use of simple rules of thumb or "ballpark" estimates for required input values is discouraged because this use is likely to lead to a significant cumulative error in performance estimates.

Automobile Mode

The required input variables for the automobile methodology are identified in the following list. These variables represent the minimum basic input data the analyst will need to provide for an analysis and were previously defined in the text associated with Exhibit 18-6:

- Demand flow rate,
- Initial queue,
- Pedestrian flow rate,
- Bicycle flow rate,
- Number of lanes,
- Number of receiving lanes,
- Turn bay length,

- Presence of on-street parking,
- Type of signal control,
- Phase sequence,
- Left-turn operational mode,
- Speed limit, and
- Area type.

Initial queue has a significant effect on delay and can vary widely among intersections and traffic movements. If it is not possible to obtain an initial queue estimate, then the analysis period should be established so that the previous period is known to have demand less than capacity and no residual queue. A multiple-period analysis may be appropriate when the duration of congestion exceeds 15 min (i.e., 0.25 h).

Several authoritative reference documents (*32*–*34*) provide useful guidelines for selecting the type of signal control, designing the phase sequence, and selecting the left-turn operational mode (i.e., permitted, protected, or protected-permitted).

Exhibit 18-28 lists default values for the automobile methodology based on national research (*35*). Some of the values listed may also be useful for the pedestrian or bicycle methodologies. The last column of this exhibit indicates "see discussion" for some variables. In these situations, the default value is described in the discussion provided in this subsection.

Many of the controller settings are specific to an actuated phase and fully actuated signal control. If pretimed control is used and the phase durations are known, the cycle length and phase duration are set to equal the known values. If pretimed control is used and the phase durations are not known, then the quick estimation method described in Chapter 31, Signalized Intersections: Supplemental, should be used to estimate the cycle length and the duration of each phase. For semiactuated control, phases with a fixed duration should have their recall mode set to "recall-to-maximum" and their maximum green limit set to the known green interval duration.

Platoon Ratio

A default value for platoon ratio can be determined from arrival type. Once the default arrival type is determined, Exhibit 18-8 is consulted to determine the equivalent default platoon ratio for input to the methodology.

In the absence of more detailed information from Chapter 17 or field measurements, a default arrival type of 3 is used for uncoordinated through movements and a default value of 4 is used for coordinated through movements. Exhibit 18-29 provides further guidance on the relationship between arrival type, street segment length, and the provision of signal coordination for through movements.

Exhibit 18-28
Default Values: Automobile Mode with Fully or Semiactuated Signal Control

Data Category	Input Data Element	Default Values
Traffic characteristics	Right-turn-on-red flow rate	0.0 veh/h
	Percent heavy vehicles	3%
	Intersection peak hour factor	If analysis period is 0.25 h and hourly data are used: Total entering volume ≥ 1,000 veh/h: 0.92 Total entering volume < 1,000 veh/h: 0.90 Otherwise: 1.00
	Platoon ratio	See discussion
	Upstream filtering adjustment factor	1.0
	Base saturation flow rate	Metropolitan area with population ≥ 250,000: 1,900 pc/h/ln Otherwise: 1,750 pc/h/ln
	Lane utilization adjustment factor	See discussion
	On-street parking maneuver rate	See discussion
	Local bus stopping rate	When buses expected to stop Central business district: 12 buses/h Non–central business district: 2 buses/h When buses not expected to stop: 0
Geometric design	Average lane width	12 ft
	Approach grade (negative for downhill conditions)	Flat approach: 0% Moderate grade on approach: 3% Steep grade on approach: 6%
Controller settings	Dallas left-turn phasing option	Dictated by local use
	Passage time	2.0 s (presence detection)
	Maximum green	Major-street through movement: 50 s Minor-street through movement: 30 s Left-turn movement: 20 s
	Minimum green	Major-street through movement: 10 s Minor-street through movement: 8 s Left-turn movement: 6 s
	Yellow change + red clearance[a]	4.0 s
	Walk	Actuated: 7.0 s Pretimed: green interval minus pedestrian clear
	Pedestrian clear	Based on 3.5-ft/s walking speed
	Phase recall	Actuated phase: No Pretimed phase: Recall to maximum
	Dual entry	Not enabled (i.e., use single entry)
	Simultaneous gap-out	Enable
Other	Analysis period duration	0.25 h
	Stop-line detector length	40 ft (presence detection)

Note: [a] Specific values of yellow change and red clearance should be determined by local guidelines or practice.

In the absence of more detailed information from Chapter 17 or field measurements, Arrival Type 3 is used for turn movements because they are typically not coordinated.

Exhibit 18-29
Progression Quality and Arrival Type

Arrival Type	Progression Quality	Signal Spacing (ft)	Conditions Under Which Arrival Type Is Likely to Occur
1	Very poor	≤1,600	Coordinated operation on a two-way street where the subject direction does not receive good progression
2	Unfavorable	>1,600–3,200	A less extreme version of Arrival Type 1
3	Random arrivals	>3,200	Isolated signals or widely spaced coordinated signals
4	Favorable	>1,600–3,200	Coordinated operation on a two-way street where the subject direction receives good progression
5	Highly favorable	≤1,600	Coordinated operation on a two-way street where the subject direction receives good progression
6	Exceptional	≤800	Coordinated operation on a one-way street in dense networks and central business districts

Lane Utilization Adjustment Factor

The default lane utilization factors described in this subpart apply to situations in which drivers randomly choose among the exclusive-use lanes on the intersection approach. The factors do not apply to special conditions (such as short lane drops or a downstream freeway on-ramp) that might cause drivers intentionally to choose their lane position on the basis of an anticipated downstream maneuver. Exhibit 18-30 provides a summary of lane utilization adjustment factors for different lane group movements and numbers of lanes.

Exhibit 18-30
Default Lane Utilization Adjustment Factors

Lane Group Movement	Number of Lanes in Lane Group (ln)	Traffic in Most Heavily Traveled Lane (%)	Lane Utilization Adjustment Factor f_{LU}
Exclusive through	1	100.0	1.000
	2	52.5	0.952
	3[a]	36.7	0.908
Exclusive left turn	1	100.0	1.000
	2[a]	51.5	0.971
Exclusive right turn	1	100.0	1.000
	2[a]	56.5	0.885

Note: [a] If a lane group has more lanes than shown in this exhibit, it is recommended that field surveys be conducted or the smallest f_{LU} value shown for that type of lane group be used.

As demand approaches capacity, the analyst may use lane utilization factors that are closer to 1.0 than those offered in Exhibit 18-30. This refinement to the factor value recognizes that a high volume-to-capacity ratio is associated with a more uniform use of the available lanes because of reduced opportunity for drivers to select their lane freely.

On-Street Parking Maneuver Rate

Exhibit 18-31 gives default values for the parking maneuver rate on an intersection approach with on-street parking. It is estimated for a distance of 250 ft back from the stop line. The calculations assume 25 ft per parking space and 80% occupancy. Each turnover (one car leaving and one car arriving) generates two parking maneuvers.

Exhibit 18-31
Default Parking Maneuver Rate

Street Type	Number of Spaces in 250 ft	Parking Time Limit (h)	Turnover Rate (veh/h)	Maneuver Rate (maneuvers/h)
Two-way	10	1	1.0	16
		2	0.5	8
One-way	20	1	1.0	32
		2	0.5	16

Automobile Mode (Coordinated-Actuated Operation)

Exhibit 18-32 lists the default values for evaluating signalized intersections that are part of a coordinated-actuated signal system. The text "see discussion" in the last column of this exhibit indicates that the default value is described in the discussion provided in this part.

Exhibit 18-32
Default Values: Automobile Mode with Coordinated-Actuated Signal Control

Data Category	Input Data Element	Default Value
Controller settings	Cycle length	See discussion
	Phase splits	See discussion
	Offset	Equal to travel time in Phase 2 direction [a]
	Offset reference	End of green for Phase 2 [a]
	Force mode	Fixed

Note: [a] Assumes that Phase 2 is the reference phase. Substitute 6 if Phase 6 is the reference phase.

Cycle Length

The cycle length used for a coordinated signal system often represents a compromise value based on intersection capacity, queue size, phase sequence, segment length, speed, and progression quality. Consideration of these factors leads to the default cycle lengths shown in Exhibit 18-33.

Exhibit 18-33
Default System Cycle Length

Average Segment Length (ft) [a]	Cycle Length by Street Class and Left-Turn Phasing (s) [b]					
	Major Arterial Street			Minor Arterial Street or Grid Network		
	No Left-Turn Phases	Left-Turn Phases on One Street	Left-Turn Phases on Both Streets	No Left-Turn Phases	Left-Turn Phases on One Street	Left-Turn Phases on Both Streets
1,300	90	120	150	60	80	120
2,600	90	120	150	100	100	120
3,900	110	120	150			

Notes: [a] Average length based on all street segments in the signal system.
[b] Selected left-turn phasing column should describe the phase sequence at the high-volume intersections in the system.

Phase Splits

If the phase splits are not known, they can be estimated by using the quick estimation method described in Chapter 31. The method can be used to estimate the effective green time for each phase on the basis of the established system cycle length. The phase split D_p is then computed by adding 4 s of lost time to the estimated effective green time (i.e., $D_p = g + 4.0$).

Nonautomobile Modes

The required input variables for the pedestrian and bicycle methodologies are identified in the following list. These variables represent the minimum basic input data the analyst will need to provide for an analysis. These variables were previously defined in the text associated with Exhibit 18-9.

- Demand flow rate of motorized vehicles,
- RTOR flow rate (pedestrian mode only),
- Permitted left-turn flow rate (pedestrian mode only),
- Pedestrian flow rate (pedestrian mode only),
- Bicycle flow rate (bicycle mode only),
- Number of lanes,
- Crosswalk length (pedestrian mode only), and
- Pedestrian signal head presence (pedestrian mode only).

The RTOR flow rate does not have a default value for application of the pedestrian methodology. This flow rate has both a notable effect on performance estimates and a wide range of possible values. The analyst is encouraged to conduct measurements at intersections for the purpose of developing local defaults for this variable.

The permitted left-turn flow rate for movements served by the permitted mode is equal to the left-turn demand flow rate. The permitted left-turn flow rate for movements served by the protected-permitted mode does not have a default value. This flow rate has both a notable effect on performance estimates and a wide range of possible values. It should be measured in the field if possible. If the analysis is dealing with future conditions or if the permitted left-turn flow rate is not known from field data, its value can be approximated as the left-turn arrival rate during the permitted period of the protected-permitted operation. This rate should equal the left-turn arrival rate during the effective red time [i.e., $q_r = (1 - P)qC/r$].

The pedestrian flow rate data consist of count data for each of five pedestrian movements at each intersection corner. These variables are shown as $v_{a,b}$, v_{ci}, v_{co}, v_{di}, and v_{do} in Exhibit 18-10.

Exhibit 18-34 lists the default values for the pedestrian and bicycle methodologies (*25–27*).

TYPES OF ANALYSIS

The automobile, pedestrian, and bicycle methodologies described in this chapter can each be used in three types (or levels) of analysis. These analysis levels are described as operational, design, and planning and preliminary engineering. The characteristics of each analysis level are described in later parts of this subsection.

Operational Analysis

Each of the methodologies is most easily applied at an operational level of analysis. At this level, all traffic, geometric, and signalization conditions are specified as input variables by the analyst. These input variables are used in the methodology to compute various performance measures.

Exhibit 18-34
Default Values:
Nonautomobile Modes

Data Category	Input Data Element	Default Value
Traffic characteristics	Intersection peak hour factor (motorized vehicles)	If analysis period is 0.25 h and hourly data are used: Total entering volume ≥ 1,000 veh/h: 0.92 Total entering volume < 1,000 veh/h: 0.90 Otherwise: 1.00
	Midsegment 85th percentile speed	Speed limit (mi/h)
	Proportion of on-street parking occupied	0.50 (if parking lane present)
Geometric design	Street width	Based on a 12-ft lane width
	Number of right-turn islands	None
	Width of outside through lane	12 ft
	Width of bicycle lane	5.0 ft (if provided)
	Width of paved outside shoulder	No parking lane: 2.0 ft (curb and gutter width) Parking lane present: 8.0 ft
	Total walkway width	Business or office land use: 9.0 ft Residential or industrial land use: 11.0 ft
	Crosswalk width	12 ft
	Corner radius	Trucks and buses in turn volume: 45 ft No trucks or buses in turn volume: 25 ft
Signal control	Walk	Actuated: 7 s Pretimed: green interval minus pedestrian clear
	Pedestrian clear	Based on 3.5-ft/s walking speed
	Rest in walk	Not enabled
	Cycle length	Based on default values determined for automobile mode
	Yellow change + red clearance [a]	4 s
	Duration of phases serving pedestrians and bicycles	Based on default values determined for automobile mode
Other	Analysis period duration	0.25 h

Note: [a] Specific values of yellow change and red clearance should be determined by local guidelines or practice.

Design Analysis

The design level of analysis has two variations. Both variations require specifying (*a*) traffic conditions and (*b*) target levels for a specified set of performance measures. One variation requires the additional specification of the signalization conditions. The methodology is then applied by using an iterative approach in which alternative geometric conditions are separately evaluated.

The second variation of the design level requires the additional specification of geometric conditions. The methodology is then applied by using an iterative approach in which alternative signalization conditions are separately evaluated.

The objective with either variation is to identify alternatives that operate at the target level of the specified performance measures (or provide a better level of performance). The analyst may then recommend the "best" alternative based on consideration of the full range of factors.

Planning and Preliminary Engineering Analysis

The planning and preliminary engineering level of analysis is intended to provide an estimate of the LOS for either a proposed intersection or an existing intersection in a future year. This level of analysis may also be used for a preliminary engineering activity to size the overall geometrics of a proposed intersection.

The level of precision inherent in planning and preliminary engineering analyses is typically lower than for operational analyses. Therefore, default values are often substituted for field-measured values of many of the input variables. Recommended default values for this purpose were described previously in this section.

The requirement for a complete description of the signal timing plan can be a burden for some planning analyses, especially when the signal control is pretimed or coordinated-actuated. The quick estimation method described in Chapter 31 can be used to estimate a reasonable timing plan, in conjunction with the aforementioned default values.

USE OF ALTERNATIVE TOOLS

General alternative tool guidance is provided in Chapters 6 and 7.

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools, and Chapter 7, Interpreting HCM and Alternative Tool Results. This section contains specific guidance for applying alternative tools to the analysis of signalized intersections. Additional information on this topic may be found in the Technical Reference Library in Volume 4.

Strengths of the Automobile Methodology

The automobile methodology described in Section 2 offers a comprehensive procedure for analyzing the performance of a signalized intersection. It models the driver–vehicle–road–signal system with reasonable accuracy for most applications. Simulation-based traffic analysis tools offer a more detailed treatment of the arrival and departure of vehicles and their interaction with the roadway and the control system. As such, some simulation tools can model the driver–vehicle–road–signal system more accurately for some applications.

The automobile methodology offers the following advantages over the use of simulation-based analysis tools:

- Its empirically calibrated saturation flow rate adjustment factors can produce an accurate estimate of saturation flow rate (simulation tools require saturation flow rate as an input variable).
- It produces a direct estimate of capacity and volume-to-capacity ratio (these measures are much more difficult to quantify with simulation).
- It produces an estimate of expected, long-run performance for a variety of measures (multiple runs and supplemental calculations are required to obtain this type of estimate with a simulation tool).

Identified Limitations of the Automobile Methodology

The limitations of the automobile methodology are identified near the end of Section 1. If any of these limitations applies to a particular situation, then alternative tools may produce more credible performance estimates. Limitations involving consideration of the impact of progression on performance are a special case that is discussed in more detail in Chapter 16, Urban Street Facilities.

Features and Performance Measures Available from Alternative Tools

This chapter provides a methodology for estimating the capacity, control delay, LOS, and back of queue associated with a lane group at a signalized intersection. Alternative tools often offer additional performance measures such as number of stops, fuel consumption, air quality, and operating costs.

Development of HCM-Compatible Performance Measures Using Alternative Tools

The LOS assessment for signalized intersections is based on control delay, which is defined as the excess travel time caused by the action of the control device (in this case, the signal).

Simulation-based analysis tools often use a definition of delay that is different from that used in the automobile methodology, especially for movements that are oversaturated at some point during the analysis. Therefore, some care must be taken in the determination of LOS when simulation-based delay estimates are used. Delay comparison among different tools is discussed in more detail in Chapter 7.

An accurate estimate of control delay may be obtained from a simulation tool by performing simulation runs with and without the control device(s) in place. The segment delay reported with no control is the delay due to geometrics and interaction between vehicles. The additional delay reported in the run with the control in place is, by definition, the control delay.

Conceptual Differences That Preclude Direct Comparison of Results

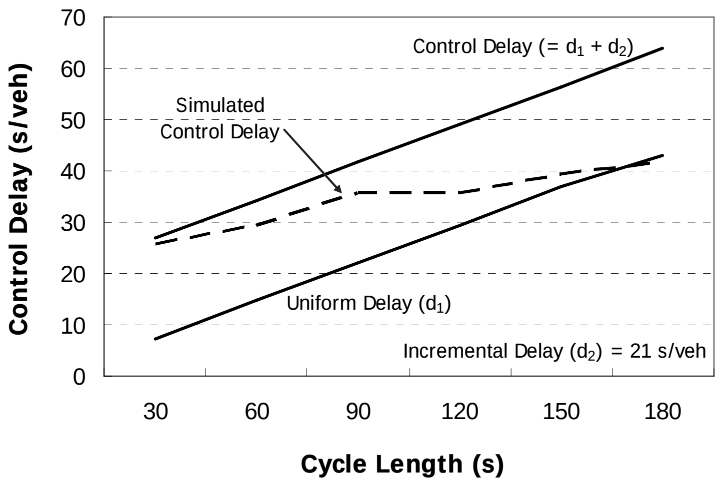
Conceptual differences in modeling approach may preclude the direct comparison of performance measures from the automobile methodology with those from alternative tools. The treatment of random arrivals is a case in point. There is a common misconception among analysts that alternative tools treat random arrivals in a similar manner.

A simple case is used to demonstrate the different ways alternative tools model random arrivals. Consider an isolated intersection with a two-phase sequence. The subject intersection approach serves only a through movement; there are no turning movements from upstream intersections or driveways. The only parameter that is allowed to vary in this example is the cycle length (all other variables are held constant).

The results of this experiment are shown in Exhibit 18-35. The two solid lines represent delay estimates obtained from the automobile methodology. Uniform delay is shown to increase linearly with cycle length. Incremental delay is constant with respect to cycle length because the volume-to-capacity ratio is constant. As a result, control delay (being the sum of the uniform and incremental delay) is also shown to increase linearly with cycle length.

Exhibit 18-35
Effect of Cycle Length on Delay

LIVE GRAPH
Click here to view



The dashed line represents the control delay estimate obtained from a simulation-based analysis tool. The simulation-based tool shows close agreement with the automobile methodology for short cycles but deviates for longer cycles. There are likely to be explainable reasons for this difference; however, the point is that such differences are likely to exist among tools. The analyst should understand the underlying modeling assumptions and limitations inherent in any tool (including the automobile methodology) when it is used. Moreover, the analyst should fully understand the definition of any performance measure used so as to interpret the results and observed trends properly.

Adjustment of Alternative Tool Parameters

For applications in which either an alternative tool or the automobile methodology can be used, some adjustment is generally required for the alternative tool if some consistency with the automobile methodology is desired. For example, the parameters that determine the capacity of a signalized approach (e.g., saturation flow rate and start-up lost time) should be adjusted to ensure that the simulated lane group (or approach) capacities match those estimated by the automobile methodology.

Step-by-Step Recommendations for Applying Alternative Tools

This part provides recommendations specifically for signalized intersection evaluation. The following steps should be taken to apply an alternative tool for signalized intersection analysis:

1. Determine whether the automobile methodology can provide a realistic assessment of the capacity and control delay for the signalized approaches of interest. The limitations stated at the end of Section 1 provide a good starting point for this assessment. If there are no conditions outside these limitations, then it should not be necessary to consider alternative tools. Otherwise, proceed with the remaining steps.

2. Select the appropriate tool in accordance with the general guidelines presented in Chapter 7.
3. Enter all available input characteristics and parameters.
4. Use the tool to evaluate the intersection. Be careful to observe the guidance provided in Chapter 7 regarding self-aggravating conditions that occur near capacity. If the tool is simulation based, then estimate the required number of runs so that the comparison is statistically valid.
5. If the documented delay definition and computational methodology used by the tool conform to the specifications set forth in Chapter 7 of this manual, then the delay estimates should be suitable for estimating the LOS. Otherwise, no such estimate should be attempted.

Sample Calculations Illustrating Alternative Tool Applications

Chapter 31 includes example problems that address the following conditions:

- Left-turn storage bay overflow,
- RTOR operation,
- Short through lanes, and
- Closely spaced intersections.

4. EXAMPLE PROBLEMS

INTRODUCTION

This part of the chapter describes the application of each of the automobile, pedestrian, and bicycle methodologies through the use of example problems. Exhibit 18-36 provides an overview of these problems. The examples focus on the operational analysis level. The planning and preliminary engineering analysis level is identical to the operational analysis level in terms of the calculations, except that default values are used when field-measured values are not available.

Exhibit 18-36
Example Problems

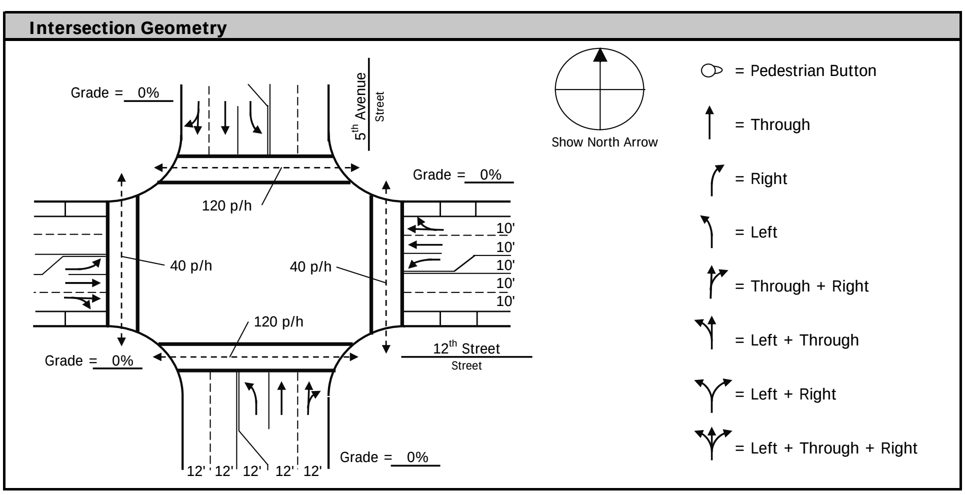
Problem Number	Description	Analysis Level
1	Automobile LOS	Operational
2	Pedestrian LOS	Operational
3	Bicycle LOS	Operational

EXAMPLE PROBLEM 1: AUTOMOBILE LOS

The Intersection

The intersection of 5th Avenue and 12th Street is an intersection of two urban arterial streets. It is shown in Exhibit 18-37.

Exhibit 18-37
Example Problem 1: Intersection Plan View



The Question

What is the motorist delay and LOS during the analysis period for each lane group and the intersection as a whole?

The Facts

The intersection's traffic, geometric, and signalization conditions are listed in Exhibit 18-38 and Exhibit 18-39.

Exhibit 18-38
Example Problem 1: Signal Conditions

Controller Data Worksheet

General Information

Analyst:	BR	Intersection:	5th Avenue/12th Street
Agency or Company:		Area Type: CBD	Phase 2: EB
Date Performed:	2/11/2010		
Analysis Time Period:	5:30 pm to 5:45 pm	Analysis Year:	2010
Filename:	C:\Documents and Settings\TexasEX3		

Phase Sequence and Left-Turn Mode

WB left (1) with WB thru (6)	EB left (5) with EB thru (2)	NB left (3) before SB thru (4)	SB left (7) before NB thru (8)
WB left permitted	EB left permitted	NB left (3) prot-perm	SB left (7) prot-perm

Phase Settings

Approach	Eastbound		Westbound		Northbound		Southbound	
Phase number		2		6	3	8	7	4
Movement		L+T+R		L+T+R	L	T+R	L	T+R
Lead/lag left-turn phase		--		--	Lead	--	Lead	--
Left-turn mode		Perm.		Perm.	Pr/Pm	--	Pr/Pm	--
Passage time, s		2.0		2.0	2.0	2.0	2.0	2.0
Maximum green, s		30		30	25	50	25	50
Minimum green, s		5		5	5	5	5	5
Yellow change, s		4.0		4.0	4.0	4.0	4.0	4.0
Red clearance, s		0.0		0.0	0.0	0.0	0.0	0.0
Walk+ ped. clear, s		19		19		21		21
Recall?:	No	No	No	No	No	No	No	No
Dual entry	No	Yes	No	Yes	No	Yes	No	Yes

Enable Simultaneous Gap-Out (check = Yes)?
Phase Group 1,2,5,6: ☑ Phase Group 3,4,7,8: ☑

Enable Dallas Left-Turn Phasing?
Phases 1,2,5,6: ☐ Phases 3,4,7,8: ☐

Protected right-turn with left-turn phase?	n.a.		n.a.		Eastbd. right		Westbd. right	
	No		No		No		No	

Phase number assignment to timers (by ring):					Controller timer ring structure:				
Ring 1:	0	2	3	4	Ring 1:	Timer 1	Timer 2	Timer 3	Timer 4
Ring 2:	0	6	7	8	Ring 2:	Timer 5	Timer 6	Timer 7	Timer 8

Exhibit 18-39
Example Problem 1: Traffic and Geometric Conditions

Movement-Specific Intersection Data Worksheet

Approach	Eastbound			Westbound			Northbound			Southbound		
Movement	L	T	R	L	T	R	L	T	R	L	T	R
Movement number	5	2	12	1	6	16	3	8	18	7	4	14
Traffic Characteristics *(Enter the volume data in all columns. For all other blue cells, enter values only if there are one or more lanes.)*												
Volume, veh/h	71	318	106	118	600	24	133	1,644	111	194	933	111
Right-turn-on-red volume, veh/h			0			0			22			33
Percent heavy vehicles, %	5	5		5	5		2	2		2	2	
Lane utilization adjustment factor	1.000	1.000		1.000	1.000		1.000	1.000		1.000	1.000	
Peak hour factor 1.00	1.00	1.00		1.00	1.00		1.00	1.00		1.00	1.00	
Start-up lost time, s	2.0	2.0		2.0	2.0		2.0	2.0		2.0	2.0	
Extension of eff. green time, s	2.0	2.0		2.0	2.0		2.0	2.0		2.0	2.0	
Platoon ratio	1.000	1.000		1.000	1.000		1.000	1.000		1.000	1.000	
Upstream filtering factor	1.00	1.00		1.00	1.00		1.00	1.00		1.00	1.00	
Pedestrian volume, p/h		120			120			40			40	
Bicycle volume, bicycles/h		0			0			0			0	
(future use)												
Initial queue, veh	0	0		0	0		0	0		0	0	
Speed limit, mph	35	35	35	35	35	35	35	35	35	35	35	35
(future use)												
Multiple-Period Analysis Counts *(if all cell values = 0, then values in the 'Volume' row above will be used for a single-period analysis)*												
Period **1** traffic count, veh												
Period **2** traffic count, veh												
Period **3** traffic count, veh												
Period **4** traffic count, veh												
Intersection Approach Characteristics *(Enter the number of lanes. For all blue cells, enter values only if there are one or more lanes.)*												
Number of lanes	1	2	0	1	2	0	1	2	0	1	2	0
Lane assignment	L	TR	n.a.	L	TR	n.a.	L	TR	n.a.	L	TR	n.a.
Average lane width, ft	10.0	10.0		10.0	10.0		12.0	12.0		12.0	12.0	
Number of receiving lanes		2			2			2			2	
Turn bay or segment length, ft	200	999		200	999		200	999		200	999	
Approach Data	*Left Side*		*Right Side*	*Left Side*		*Right Side*	*Left Side*		*Right Side*	*Left Side*		*Right Side*
Parking present?	No		Yes	No		Yes	No		No	No		No
Parking maneuvers, maneuvers/h	0		5	0		5	0		0	0		0
Bus stopping rate, buses/h			0			0			0			0
Approach grade, %	0	0	0	0	0	0	0	0	0	0	0	0
Detection Data *(Enter values only if there are one or more lanes.)*												
Stop line detector length, ft	40	40	n.a.	40	40	n.a.	40	40	n.a.	40	40	n.a.
(future use)												

The intersection is located in a central business district–type environment. Adjacent signals are somewhat distant so the intersection is operated by using fully actuated control. Vehicle arrivals to each approach are characterized as "random" and are described by using a platoon ratio of 1.0.

The left-turn movements on the north–south street operate under protected-permitted control and lead the opposing through movements (i.e., a lead–lead phase sequence). The left-turn movements on the east–west street operate as permitted.

All intersection approaches have a 200-ft left-turn bay, an exclusive through lane, and a shared through and right-turn lane. The average width of the traffic lanes on the east–west street is 10 ft. The average width of the traffic lanes on the north–south street is 12 ft.

Crosswalks are provided on each intersection leg. A two-way flow rate of 120 p/h is estimated to use each of the east–west crosswalks and a two-way flow rate of 40 p/h is estimated to use each of the north–south crosswalks.

On-street parking is present on the east–west street. It is estimated that parking maneuvers on each intersection approach occur at a rate of 5 maneuvers/h during the analysis period.

The speed limit is 35 mi/h on each intersection approach. The analysis period is 0.25 h. There is no initial queue for any movement.

As noted in the next section, none of the lane groups at the intersection has two or more exclusive lanes. For this reason, the saturation flow rate adjustment factor for lane utilization is equal to 1.0 for all approaches. Any unequal lane use that may occur due to the shared through and right-turn lane groups will be accounted for in the lane group flow rate calculation, as described in Chapter 31.

Outline of Solution

Movement-Based Data

Exhibit 18-40 provides a summary of the analysis of the individual traffic movements at the intersection. The movement numbers shown follow the numbering convention in Exhibit 18-2.

Exhibit 18-40
Example Problem 1: Movement-Based Output Data

Movement:	EB L 5	EB T 2	EB R 12	WB L 1	WB T 6	WB R 16	NB L 3	NB T 8	NB R 18	SB L 7	SB T 4	SB R 14
Volume, veh/h	71	318	106	118	600	24	133	1,644	89	194	933	78
Initial Queue, veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj. Factor (A_pbT)	0.999		0.878	0.976		0.878	0.999		0.976	1.000		0.977
Parking, Bus Adj. Factors (f_bb x f_p)	1.000	1.000	0.875	1.000	1.000	0.875	1.000	1.000	1.000	1.000	1.000	1.000
Adjusted Sat. Flow Rate, veh/h/ln	1,629	1,629	1,629	1,629	1,629	1,629	1,676	1,676	1,676	1,676	1,676	1,676
Lanes	1	2	0	1	2	0	1	2	0	1	2	0
Lane Assignment	L	TR	n.a.	L	TR	n.a.	L	TR	n.a.	L	TR	n.a.
Capacity, veh/h	147.23	629.27	201.44	205.81	853.60	34.08	326.46	1,545.51	83.10	224.96	1,604.59	134.14
Proportion Arriving On Green	0.294	0.294	0.294	0.294	0.294	0.294	0.061	0.491	0.491	0.097	0.527	0.527
Approach Volume, veh/h		495			742			1,866			1,205	
Approach Delay, s/veh		32.553			37.432			71.532			19.828	

Note: n.a. = not applicable

Two saturation flow rate adjustment factors are shown in Exhibit 18-40. One factor is the pedestrian–bicycle adjustment factor. This factor is used to estimate the saturation flow rate for the turn movement in a lane group. The "parking, bus adjustment factor" represents the product of the parking adjustment factor and the bus blockage adjustment factor. This combined factor is computed separately for the lane group that is adjacent to the parking or bus stop.

The adjusted saturation flow rate represents the saturation flow rate for all lane groups on the approach. It reflects the combined effect of lane width, heavy-vehicle presence, grade, and area type. The effect of pedestrians, bicycles, parking, bus blockage, lane utilization, right-turn maneuvers, and left-turn maneuvers is calculated separately at a later stage of the analysis because their values are influenced by signal timing, lane group demand flow rate, and lane group location (adjacent to parking or not, etc.). As such, these factors are internal to the iterative sequence of calculations used to estimate signal phase duration.

Capacity for a movement is computed by using the movement volume proportion in the lane group, lane group saturation flow rate, and corresponding phase duration. This variable represents the capacity of the movement, regardless of whether it is served in an exclusive lane or in a shared lane. If the movement is served in a shared lane, then the movement capacity represents the portion of the lane group capacity available to the movement, as distributed in proportion to the flow rate of the movements served by the associated lane group.

The last two rows in Exhibit 18-40 represent summary statistics for the approach. The approach volume represents the sum of the three movement volumes. Approach delay is computed as volume-weighted average for the lane groups served on an intersection approach.

Timer-Based Phase Data

Exhibit 18-41 provides a summary of the output data by using a signal controller perspective. The controller has eight timing functions (or timers), with Timers 1 to 4 representing Ring 1 and Timers 5 to 8 representing Ring 2. The ring structure and phase assignments were previously shown at the bottom of Exhibit 18-38. Timers 1 and 5 are not used at this intersection.

Exhibit 18-41
Example Problem 1: Timer-Based Phase Output Data

Timer Data

Timer:	1	2 EB L.T.T+R	3 NB L	4 SB T.T+R	5	6 WB L.T.T+R	7 SB L	8 NB T.T+R
Assigned Phase		2	3	4		6	7	8
Case No		6	1	4		6	1	4
Phase Duration (G+Y+Rc), s		34.00	10.21	57.66		34.00	13.87	54.00
Change Period (Y+Rc), s		4.00	4.00	4.00		4.00	4.00	4.00
Max. Allowable Headway (MAH), s		3.44	3.13	3.06		3.44	3.13	3.06
Maximum Green Setting (Gmax), s		30.00	25.00	50.00		30.00	25.00	50.00
Max. Queue Clearance Time (g_c+I1), s		31.10	6.16	23.29		29.51	9.61	52.00
Green Extension Time (g_e), s		0.000	0.199	7.831		0.238	0.296	0.000
Probability of Phase Call (p_c)		1.000	0.977	1.000		1.000	0.996	1.000
Probability of Max Out (p_x)		1.000	0.000	0.179		1.000	0.000	1.000

Equilibrium Cycle Length, s: 102

The timing function construct is essential in modeling a ring-based signal controller. *Timers* always occur in the same numeric sequence (i.e., 1 then 2 then 3 then 4 in Ring 1; 5 then 6 then 7 then 8 in Ring 2). The practice of associating movements to phases (e.g., the major-street through movement to Phase 2) coupled with the occasional need for lagging left-turn phases and split phasing creates the situation in which *phases* do not always time in sequence. For example, with a lagging left-turn phase sequence, major-street through Phase 2 times first and then major-street left-turn Phase 1 times second.

The modern controller accommodates the assignment of phases to timing functions by allowing the ring structure to be redefined manually or by time-of-day settings. Specification of this structure is automated in the computational engine by assigning phases to timers.

The methodology is based on modeling *timers*, not by directly modeling movements or phases. The methodology converts movement and phase input data into timer input data. It then models controller response to these inputs and computes timer duration and related performance measures.

The signalized intersection in this example problem has a lead–lead left-turn phase sequence on the north–south street. Hence, the timer numbers for this street are the same as the phase numbers, which are the same as the movement numbers (e.g., the northbound left-turn Movement 3 is associated with Phase 3, which is assigned to Timer 3). In contrast, the east–west street does not have left-turn phases, so one timer and one phase are used to serve all movements on a given approach.

The case number shown in Exhibit 18-41 is used as a single variable descriptor of each possible combination of left-turn mode and lane-group type (i.e., shared or exclusive). An understanding of this variable is not needed to interpret the output data.

The phase duration shown in the exhibit represents the estimated average phase duration during the analysis period. It represents the sum of the green, yellow change, and red clearance intervals. For Timer 2 (i.e., Phase 2), the average green interval duration is 30 s (= 34.00 – 4.00).

The durations of Phases 2, 3, and 4 add to the average cycle length of 101.87 s (= 34.00 + 10.21 + 57.66). Similarly, the durations of Phases 6, 7, and 8 add to the cycle length.

The cycle length is described in Exhibit 18-41 to be the "equilibrium" cycle length. The equilibrium cycle length is the average cycle length when all phase durations are dictated by traffic demand. However, the duration of several phases at this intersection is constrained by their maximum green limit. As such, the cycle length shown is not truly an equilibrium cycle length for this particular intersection.

The maximum green setting is input by the analyst. If the intersection were operated as coordinated-actuated, the "equivalent" maximum green setting would be shown here. It would be computed from the input phase splits and would reflect the specified force mode.

The maximum queue clearance time represents the largest queue clearance time of all lane groups served by the phase. Queue clearance time represents the time between the start of the green interval and the end of the queue service period. It is determined from the queue accumulation polygon. It includes the start-up lost time.

The maximum allowable headway, maximum green, and maximum queue clearance time apply only to actuated phases. They are not relevant to calculation of coordinated phase duration.

The green extension time represents the time the green interval is extended by arriving vehicles. This value is 0.0 s for two timers because they terminate by extension to their maximum limit (i.e., max-out).

The probability of a phase call represents the probability that one or more vehicles will place a call for service on the associated timer. The probability of max-out represents the probability that the phase will extend to the maximum green setting and terminate, perhaps leaving some unserved vehicles on the intersection approach.

Timer-Based Movement Data

Exhibit 18-42 summarizes the output for the vehicle movements assigned to each timer. Separate sections of output are shown in the exhibit for the left-turn, through, and right-turn movements. The assigned movement row identifies the movement (previously identified in Exhibit 18-40) assigned to each timer.

Exhibit 18-42
Example Problem 1: Timer-Based Movement Output Data

Timer Data								
Timer:	1	2 EB L.T.T+R	3 NB L	4 SB T.T+R	5	6 WB L.T.T+R	7 SB L	8 NB T.T+R
Left-Turn Movement Data								
Assigned Movement		5	3			1	7	
Mvmt. Sat Flow, veh/h		696.73	1,592.65			818.40	1,592.65	
Through Movement Data								
Assigned Movement		2		4		6		8
Mvmt. Sat Flow, veh/h		2,136.77		3,046.34		2,898.49		3,148.76
Right-Turn Movement Data								
Assigned Movement		12		14		16		18
Mvmt. Sat Flow, veh/h		684.02		254.67		115.71		169.31

The saturation flow rate shown in Exhibit 18-42 represents the saturation flow rate computed for the movement. For through movements in exclusive lanes, the movement saturation flow rate is equal to the number of through lanes times the adjusted saturation flow rate, times the pedestrian–bicycle adjustment factor, times the combined parking–bus blockage adjustment factor. For turn movements in exclusive lanes, the calculation is similar except that the left-turn (or right-turn) adjustment factor is also applied.

For turn movements that share a lane with a through movement, the saturation flow rate for the lane group is computed by using the procedure described in Chapter 31. The movement saturation flow rate represents the portion of the lane group saturation flow rate available to the movement, as distributed in proportion to the flow rate of the movements served by the lane group. To illustrate this point, consider Timer 4. It has a shared-lane lane group with 15.7% right-turning vehicles, 84.3% through vehicles, and a saturation flow rate of 1,624.5 veh/h/ln. The turn movement saturation flow rate is 254.67 veh/h (= 0.157 × 1,624.5). The through movement saturation flow rate in this shared lane is 1,369.8 veh/h (= 0.843 × 1,624.5). The through movement is also served by one exclusive through lane with a saturation flow rate of 1,676.5 veh/h. Thus, the total through-movement saturation flow rate is 3,046.3 veh/h (= 1,369.8 + 1,676.5). The individual lane group saturation flow rates used in this example were obtained from the lane group data described in the next few sections.

Timer-Based Left Lane Group Data

Exhibit 18-43 summarizes the output for the "left" lane group associated with an intersection approach. Each left lane group includes the left-turn movements when they exist on an intersection approach. A left lane group will also contain all the output data for a single-lane approach, regardless of whether a left-turn movement exists.

The "lane assignment" row indicates the lane groups served by the timer (e.g., L, left turn; T, through; R, right turn). The letter "L" is shown for Timers 2 and 6 as a reminder that the timer is serving a left-turn lane group. Other letter combinations are possible. For example, "L+T" indicates the timer is serving a lane group consisting of a shared lane serving left-turn and through movements. A "L+T+R" sequence indicates a single-lane approach serving all movements.

Exhibit 18-43
Example Problem 1: Timer-Based Left Lane Group Output Data

Timer Data								
Timer:	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
		EB	**NB**	**SB**		**WB**	**SB**	**NB**
		L.T.T+R	**L**	**T.T+R**		**L.T.T+R**	**L**	**T.T+R**
Left Lane Group Data								
Assigned Movement		5	3			1	7	
Lane Assignment		L	L (Pr/Pm)			L	L (Pr/Pm)	
Lanes in Group		1	1			1	1	
Group Volume (v), veh/h		71.0	133.0			118.0	194.0	
Group Sat. Flow (s), veh/h/ln		696.7	1,592.6			818.4	1,592.6	
Queue Serve Time (g_s), s		10.289	4.160			14.328	7.613	
Cycle Queue Clear Time (g_c), s		29.097	4.160			27.508	7.613	
*Perm LT Sat Flow Rate (s_l), veh/h/l		696.7	499.3			818.4	250.4	
*Shared LT Sat Flow (s_sh), veh/h/ln		0.0	0.0			0.0	0.0	
*Perm LT Eff. Green (g_p), s		30.00	50.00			30.00	55.31	
*Perm LT Serve Time (g_u), s		11.19	32.37			16.82	0.00	
*Perm LT Que Serve Time (g_ps), s		10.29	6.40			14.33	0.00	
*Time to First Blk (g_f), s		0.00	0.00			0.00	0.00	
*Serve Time pre Blk (g_fs), s		0.00	0.00			0.00	0.00	
*Proportion LT Inside Lane (P_L)		1.000	1.000			1.000	1.000	
Lane Group Capacity (c), veh/h		147.2	326.5			205.8	225.0	
Volume-to-Capacity Ratio (X)		0.482	0.407			0.573	0.862	
Available Capacity (c_a), veh/h		147.2	620.2			205.8	461.5	
Upstream Filter Factor (I)		1.000	1.000			1.000	1.000	
Uniform Delay (d1), s/veh		44.936	13.243			41.483	30.229	
Incremental Delay (d2), s/veh		0.910	0.304			2.496	3.791	
Initial Queue Delay (d3), s/veh		0.000	0.000			0.000	0.000	
Control Delay (d), s/veh		45.846	13.547			43.979	34.020	
First-Term Queue (Q1), veh/ln		1.75	1.39			2.83	2.98	
Second-Term Queue (Q2), veh/ln		0.04	0.03			0.14	0.24	
Third-Term Queue (Q3), veh/ln		0.00	0.00			0.00	0.00	
Percentile bk-of-que factor (f_B%)		1.00	1.00			1.00	1.00	
Percentile Back of Queue (Q%), veh/ln		1.78	1.42			2.97	3.22	
Percentile Storage Ratio (RQ%)		0.232	0.180			0.386	0.409	
Initial Queue (Qb), veh		0.0	0.0			0.0	0.0	
Final (Residual) Queue (Qe), veh		0.0	0.0			0.0	0.0	
Saturated Delay (ds), s/veh		0.000	0.000			0.000	0.000	
Saturated Queue (Qs), veh		0.00	0.00			0.00	0.00	
Saturated Capacity (cs), veh/h		0.0	0.0			0.0	0.0	
Initial Queue Clear Time (tc), h		0.000	0.000			0.000	0.000	

The lane assignment row also indicates the operational mode for the left-turn movements. "Prot" indicates a protected left-turn mode. "Pr/Pm" indicates a protected-permitted left-turn mode. Other designations with the letter "L" indicate either a permitted left-turn mode or split phasing.

The rows listed in Exhibit 18-43 that start with "queue serve time" and end with "uniform delay" correspond to variables that are computed from the queue accumulation polygon.

The permitted left-turn saturation flow rate represents the filtering flow rate of a permitted left-turn movement. Equations for computing this flow rate and the other variables identified with an asterisk (*) are described in Chapter 31.

The shared left-turn saturation flow rate is the saturation flow rate of a shared left-turn and through lane during the period after the first blocking left-turning vehicle arrives but before the queue service ends. This flow rate is applicable only when the opposing approach has one traffic lane. It reflects the opportunities to serve the subject approach that are created by left-turning vehicles in the opposing lane.

The permitted left-turn effective green time represents the time available for permitted left-turn movement. In general, it is the time in the opposing through movement phase that is associated with a permissive green ball signal indication. Its duration can vary with phase sequence and timing.

The permitted left-turn service time represents the time required to serve the left-turn queue. This time occurs during the permitted left-turn effective green time but after the conflicting queue clears. It exists for phases that operate in the permitted mode or in the protected-permitted mode.

The time to first block applies to a lane group with a shared lane and a left-turn movement that operates in the permitted or protected-permitted mode. It represents the time from the start of the through phase until the first left-turning vehicle arrives at the stop line and stops to wait for an acceptable gap in oncoming traffic.

The queue service time before the first block (i.e., serve time pre blk) represents the queue service time for a stream of through movements in a shared left-turn and through lane. If the left-turn flow rate is low, the time to first block may occur well into the phase. In this case, it is possible that the queue of through vehicles in the shared lane will be served before the first left-turning vehicle arrives. This variable applies only to lane groups with a shared left-turn lane.

The proportion of left-turning vehicles in the inside lane represents the distribution of vehicles in the left-lane group. If a left-turn bay exists, then the proportion equals 1.0. If the lane group is shared by left-turn and through movements, then the proportion can vary between 0.0 and 1.0. If it is 1.0, then the shared lane operates as an exclusive left-turn lane.

Uniform delay represents the area under the queue accumulation polygon. This polygon is based on an average arrival rate during the green indication and an average arrival rate during the red indication. As such, it reflects the effect of progression on the delay estimate.

The available capacity is computed for all actuated phases and noncoordinated phases. It is computed by using the maximum green setting for the phase. For coordinated phases, the available capacity is computed by using the average effective green time.

The incremental delay is computed by using the incremental delay equation. For actuated phases, it uses available capacity to estimate the incremental delay factor k. For coordinated phases and phases set to “recall-to-maximum,” it uses a factor of 0.50.

The first-term queue is a back-of-queue estimate that is obtained from an arrival–departure polygon. This polygon is based on the specification of arrival rates during the red and green intervals. As such, it reflects the effect of progression on first-term queue size. The procedure for developing this polygon is described in Chapter 31.

The second-term queue is computed as a derivative of the incremental delay estimate. It represents the average number of vehicles in queue each cycle due to random variation in arrivals plus those vehicles in queue due to oversaturation during the analysis period.

The queue storage ratio represents the ratio of the back-of-queue size to the available storage length. In general, this ratio can be computed for turn bays and through lanes; however, it is computed only for the left-turn bays in this example. A value of 0.0 indicates that no turn vehicles are queued in the bay. A value of 1.0 or more indicates that the queue completely fills the bay at some point during the cycle.

The initial queue reflects the input initial queue value when a single analysis period is evaluated. In contrast, it reflects the residual queue from the previous analysis period for the second and subsequent analysis periods of a multiple-period analysis.

The saturated delay, queue, and capacity data reflect the output from a complete (and separately computed) intersection analysis. For this separate analysis, lane groups with an initial queue will have their demand flow rate adjusted so that volume equals lane group capacity. The saturated delay equals the uniform delay computed for this "saturated" condition. Similarly, the "saturated" queue equals the first-term queue for the saturated condition.

The initial queue clear time indicates the time when the last vehicle that arrives at an overflow queue during the analysis period clears the intersection (measured from the start of the analysis period).

Timer-Based Middle Lane Group Data

Exhibit 18-44 provides a summary of the output for the "middle" lane group associated with an intersection approach. This lane group is used when one or more exclusive lanes serve through vehicles on an intersection approach. The explanation of the various output statistics is the same as that previously given for the left lane groups.

In Exhibit 18-44, the exclusive through lane served by Timer 8 has a volume-to-capacity ratio that slightly exceeds 1.0. This condition results in a large value of control delay (= 73.6 s/veh) and a final (i.e., residual) queue size of 11.8 veh. The last vehicle to arrive at this queue during the analysis period will depart the intersection 0.264 h after the *start* of the 0.25-h analysis period.

Timer-Based Right Lane Group Data

Exhibit 18-45 summarizes the output for the "right" lane group associated with an intersection approach. This lane group is used when there are two or more lanes on an intersection approach and a through or right-turn movement is present. A lane that is shared by the right-turn and through movements is always shown in the right lane group. The explanation of the various output statistics is the same as that previously given for the left lane groups.

The protected right-turn saturation flow rate row is used when the right-turn movement is provided a green arrow indication concurrently with its complementary left-turn phase on the cross street. This flow rate represents the saturation flow rate during the green arrow. Similarly, the protected right-turn effective green time equals the effective green time coincident with the green arrow indication. This operation is not provided at the subject intersection, so the values for these two variables equal 0.0.

Exhibit 18-44
Example Problem 1: Timer-Based Middle Lane Group Output Data

Timer Data								
Timer:	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
		EB	**NB**	**SB**		**WB**	**SB**	**NB**
		L.T.T+R	**L**	**T.T+R**		**L.T.T+R**	**L**	**T.T+R**
Middle Lane Group Data								
Assigned Movement		2		4		6		8
Lane Assignment		T		T		T		T
Lanes in Group		1		1		1		1
Group Volume (v), veh/h		239.2		513.4		336.6		870.1
Group Sat. Flow (s), veh/h/ln		1,628.6		1,676.5		1,628.6		1,676.5
Queue Serve Time (g_s), s		12.376		21.284		18.724		50.000
Cycle Queue Clear Time (g_c), s		12.376		21.284		18.724		50.000
Lane Group Capacity (c), veh/h		479.6		883.0		479.6		822.9
Volume-to-Capacity Ratio (X)		0.499		0.581		0.702		1.057
Available Capacity (c_a), veh/h		479.6		883.0		479.6		822.9
Upstream Filter Factor (I)		1.000		1.000		1.000		1.000
Uniform Delay (d1), s/veh		29.717		16.445		31.956		25.934
Incremental Delay (d2), s/veh		0.299		0.649		3.876		47.658
Initial Queue Delay (d3), s/veh		0.000		0.000		0.000		0.000
Control Delay (d), s/veh		30.017		17.094		35.832		73.592
First-Term Queue (Q1), veh/ln		4.73		7.61		7.15		18.26
Second-Term Queue (Q2), veh/ln		0.04		0.16		0.52		10.89
Third-Term Queue (Q3), veh/ln		0.00		0.00		0.00		0.00
Percentile bk-of-que factor (f_B%)		1.00		1.00		1.00		1.00
Percentile Back of Queue (Q%), veh/ln		4.77		7.77		7.67		29.15
Percentile Storage Ratio (RQ%)		0.124		0.198		0.200		0.741
Initial Queue (Qb), veh		0.0		0.0		0.0		0.0
Final (Residual) Queue (Qe), veh		0.0		0.0		0.0		11.8
Saturated Delay (ds), s/veh		0.000		0.000		0.000		0.000
Saturated Queue (Qs), veh		0.00		0.00		0.00		0.00
Saturated Capacity (cs), veh/h		0.0		0.0		0.0		0.0
Initial Queue Clear Time (tc), h		0.000		0.000		0.000		0.264

Exhibit 18-45
Example Problem 1: Timer-Based Right Lane Group Output Data

Timer Data								
Timer:	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
		EB	**NB**	**SB**		**WB**	**SB**	**NB**
		L.T.T+R	**L**	**T.T+R**		**L.T.T+R**	**L**	**T.T+R**
Right Lane Group Data								
Assigned Movement		12		14		16		18
Lane Assignment		T+R		T+R		T+R		T+R
Lanes in Group		1		1		1		1
Group Volume (v), veh/h		184.8		497.6		287.4		862.9
Group Sat. Flow (s), veh/h/ln		1,192.2		1,624.5		1,385.6		1,641.6
Queue Serve Time (g_s), s		13.179		21.285		18.808		50.000
Cycle Queue Clear Time (g_c), s		13.179		21.285		18.808		50.000
*Prot RT Sat Flow Rate (s_R), veh/h/ln		0.000		0.000		0.000		0.000
*Prot RT Eff. Green (g_R), s		0.000		0.000		0.000		0.000
*Proportion RT Outside Lane (P_R)		0.574		0.157		0.084		0.103
Lane Group Capacity (c), veh/h		351.1		855.7		408.1		805.7
Volume-to-Capacity Ratio (X)		0.526		0.581		0.704		1.071
Available Capacity (c_a), veh/h		351.1		855.7		408.1		805.7
Upstream Filter Factor (I)		1.000		1.000		1.000		1.000
Uniform Delay (d1), s/veh		30.001		16.445		31.986		25.934
Incremental Delay (d2), s/veh		0.729		0.670		4.631		52.458
Initial Queue Delay (d3), s/veh		0.000		0.000		0.000		0.000
Control Delay (d), s/veh		30.729		17.116		36.617		78.392
First-Term Queue (Q1), veh/ln		3.68		7.37		6.11		17.88
Second-Term Queue (Q2), veh/ln		0.07		0.16		0.52		11.74
Third-Term Queue (Q3), veh/ln		0.00		0.00		0.00		0.00
Percentile bk-of-que factor (f_B%)		1.00		1.00		1.00		1.00
Percentile Back of Queue (Q%), veh/ln		3.76		7.53		6.64		29.62
Percentile Storage Ratio (RQ%)		0.098		0.192		0.173		0.753
Initial Queue (Qb), veh		0.0		0.0		0.0		0.0
Final (Residual) Queue (Qe), veh		0.0		0.0		0.0		14.3
Saturated Delay (ds), s/veh		0.000		0.000		0.000		0.000
Saturated Queue (Qs), veh		0.00		0.00		0.00		0.00
Saturated Capacity (cs), veh/h		0.0		0.0		0.0		0.0
Initial Queue Clear Time (tc), h		0.000		0.000		0.000		0.268

Results

A comparison of the lane-group volumes in Exhibit 18-43, Exhibit 18-44, and Exhibit 18-45 indicates the extent to which drivers are expected to distribute themselves among the lane groups on each intersection approach. For example, Timer 2 serves three lane groups on the eastbound approach. The left lane group is an exclusive lane and serves all left-turn movements. The middle lane group serves 239 veh/h of the 318 veh/h in the through movement (i.e., about 75%). The right lane group serves the remaining through vehicles (i.e., 79 veh/h) and the right-turning vehicles (106 veh/h) for a total flow rate of 185 veh/h. There are fewer vehicles in the right lane group (i.e., 185 versus 239) because some through drivers choose the middle lane to avoid any possible delay that might be incurred by the presence of right-turning vehicles in the outside lane.

Exhibit 18-46 summarizes the delay for each lane group, approach, and the intersection as a whole. It also provides the volume-to-capacity ratio and LOS for each lane group. The delay varies widely among lane groups, as does the LOS. The northbound through and right-turn movements have the highest delay and a LOS F condition.

Exhibit 18-46
Example Problem 1: Performance Measure Summary

Group:	EB Left L	EB Middle T	EB Right T+R	WB Left L	WB Middle T	WB Right T+R	NB Left L (Pr/Pm)	NB Middle T	NB Right T+R	SB Left L (Pr/Pm)	SB Middle T	SB Right T+R
Lane Group Summary												
Group Volume (v), veh/h	71.0	239.2	184.8	118.0	336.6	287.4	133.0	870.1	862.9	194.0	513.4	497.6
Volume-to-Capacity Ratio (X)	0.482	0.499	0.526	0.573	0.702	0.704	0.407	1.057	1.071	0.862	0.581	0.581
Control Delay (d), s/veh	45.846	30.017	30.729	43.979	35.832	36.617	13.547	73.592	78.392	34.020	17.094	17.116
Level of Service	D	C	C	D	D	D	B	F	F	C	B	B
Approach Summary												
Approach Volume, veh/h		495.0			742.0			1866.0			1205.0	
Approach Delay, s/veh		32.553			37.432			71.532			19.828	
Level of Service		C			D			E			B	
Intersection Summary												
Entering Volume, veh/h	4308.0											
Control Delay, s/veh	46.717											
Level of Service	D											

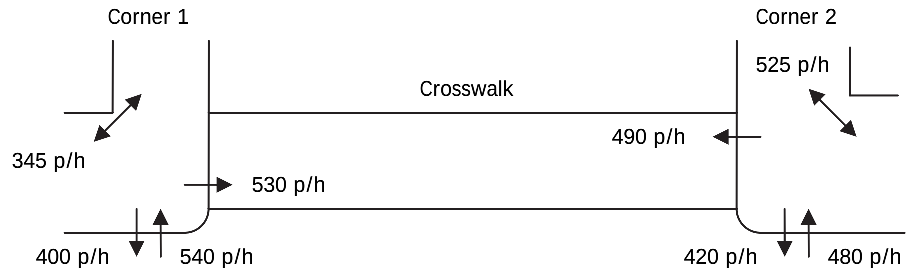
The fact that several phases are terminating by max-out and that the northbound through and right-turn movements are congested (i.e., Timer 8) suggests that some improvements could be made at this intersection. Simply increasing the maximum green settings is not a solution and, in fact, increases the overall delay and queue size for most lane groups. Physical changes to the intersection geometry to increase capacity could be considered.

EXAMPLE PROBLEM 2: PEDESTRIAN LOS

The Intersection

The pedestrian crossing of interest crosses the north leg at a signalized intersection. The north–south street is the minor street and the east–west street is the major street. The intersection serves all north–south traffic concurrently (i.e., no left-turn phases) and all east–west traffic concurrently. The signal has an 80-s cycle length. The crosswalk and intersection corners that are the subject of this example problem are shown in Exhibit 18-47.

Exhibit 18-47
Example Problem 2: Pedestrian Flow Rates



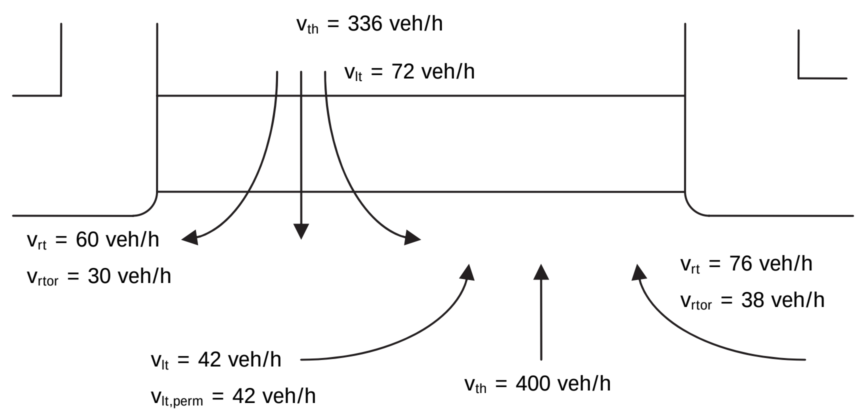
The Question

What is the pedestrian LOS for the crossing?

The Facts

Pedestrian flow rates are shown in Exhibit 18-47. Vehicular flow rates are shown in Exhibit 18-48.

Exhibit 18-48
Example Problem 2: Vehicular Demand Flow Rates



In addition, the following facts are known about the crosswalk and the intersection corners:

Major street: Phase duration, $D_{p,mj} = 48$ s
Yellow change interval, $Y_{mj} = 4$ s
Red clearance interval, $R_{mj} = 1$ s
Walk setting, $Walk_{mj} = 7$ s
Pedestrian clear setting, $PC_{mj} = 8$ s
Four traffic lanes (no turn bays)

Minor street: Phase duration, $D_{p,mi} = 32$ s
Yellow change interval, $Y_{mi} = 4$ s
Red clearance interval, $R_{mi} = 1$ s
Walk setting, $Walk_{mi} = 7$ s
Pedestrian clear setting, $PC_{mi} = 13$ s
Two traffic lanes (no turn bays)
85th percentile speed at a midsegment location, $S_{85,mi} = 35$ mi/h

Corner 1:	Total walkway width, $W_a = W_b = 16$ ft
	Corner radius, $R = 15$ ft
Corner 2:	Total walkway width, $W_a = W_b = 18$ ft
	Corner radius, $R = 15$ ft
Other data:	No right-turn channelizing islands provided on any corner
	Effective crosswalk width, $W_c = 16$ ft
	Crosswalk length, $L_c = 28$ ft
	Walking speed, $S_p = 4$ ft/s
	Pedestrian signal indications are provided for each crosswalk
	Rest-in-walk mode is not used for any phase

Comments

On the basis of the variable notation in Exhibit 18-25, the subject crosswalk is "Crosswalk C" because it crosses the minor street. The outbound pedestrian flow rate v_{co} at Corner 1 equals inbound flow rate v_{ci} at Corner 2, and the inbound flow rate v_{ci} at Corner 1 equals the outbound flow rate v_{co} at Corner 2.

Outline of Solution

First, the circulation area is calculated for both corners. Next, the circulation area is calculated for the crosswalk. The street corner and crosswalk circulation areas are then compared with the qualitative descriptions of pedestrian space listed in Exhibit 18-24.

Pedestrian delay and the pedestrian LOS score are then calculated for the crossing. Finally, LOS for the crossing is determined on the basis of the computed score and the threshold values in Exhibit 18-5.

Computational Steps

Step 1: Determine Street Corner Circulation Area

A. Compute Available Time–Space

For Corner 1, the available time–space is computed with Equation 18-52.

$$TS_{\text{corner}} = C(W_a W_b - 0.215R^2)$$

$$TS_{\text{corner}} = (80)[(16)(16) - 0.215(15)^2]$$

$$TS_{\text{corner}} = 16{,}610 \text{ ft}^2\text{-s}$$

B. Compute Holding-Area Waiting Time

Because pedestrian signal indications are provided and rest-in-walk is not enabled, the effective walk time for the phase serving the major street is computed with Equation 18-49.

$$g_{\text{Walk},mj} = Walk_{mj} + 4.0$$

$$g_{\text{Walk},mj} = 7.0 + 4.0 = 11 \text{ s}$$

The number of pedestrians arriving at the corner during each cycle to cross the minor street is computed with Equation 18-54.

$$N_{co} = \frac{v_{co}}{3{,}600} C$$

$$N_{co} = \frac{530}{3{,}600}(80) = 11.8 \text{ p}$$

The total time spent by pedestrians waiting to cross the minor street during one cycle is then calculated with Equation 18-53.

$$Q_{tco} = \frac{N_{co}(C - g_{\text{Walk},mj})^2}{2C}$$

$$Q_{tco} = \frac{(11.8)(80-11)^2}{2(80)} = 350.5 \text{ p-s}$$

By the same procedure, the total time spent by pedestrians waiting to cross the major street during one cycle (Q_{tdo}) is found to be 264.5 p-s.

C. Compute Circulation Time–Space

The circulation time–space is found by using Equation 18-58.

$$TS_c = TS_{\text{corner}} - [5.0(Q_{tdo} + Q_{tco})]$$

$$TS_c = 16{,}610 - [5.0\ (350.5 + 264.5)] = 13{,}535 \text{ ft}^2\text{-s}$$

D. Compute Pedestrian Corner Circulation Area

The total number of circulating pedestrians is computed with Equation 18-60.

$$N_{tot} = \frac{v_{ci} + v_{co} + v_{di} + v_{do} + v_{a,b}}{3{,}600} C$$

$$N_{tot} = \frac{490 + 530 + 540 + 400 + 345}{3{,}600}(80) = 51.2 \text{ p}$$

Finally, the corner circulation area per pedestrian is calculated with Equation 18-59.

$$M_{\text{corner}} = \frac{TS_c}{4.0 N_{tot}}$$

$$M_{\text{corner}} = \frac{13{,}535}{4.0(51.2)} = 66.1 \text{ ft}^2/\text{p}$$

By following the same procedure, the corner circulation area per pedestrian for Corner 2 is found to be 87.6 ft^2/p. According to the qualitative descriptions provided in Exhibit 18-24, pedestrians at both corners will have the ability to move in the desired path, with no need to alter their movements to avoid conflicts.

Step 2: Determine Crosswalk Circulation Area

The analysis conducted in this step describes the circulation area for pedestrians in the subject crosswalk.

A. Establish Walking Speed

As given in the "facts" section, the average walking speed is determined to be 4.0 ft/s.

B. Compute Available Time–Space

Rest-in-walk is not enabled, so the pedestrian service time g_{ped} is estimated to equal the sum of the walk and pedestrian clear settings. The time–space available in the crosswalk is found with Equation 18-61.

$$TS_{cw} = L_c W_c \, g_{\text{Walk},mj}$$

$$TS_{cw} = (28)(16)(11) = 4{,}928 \text{ ft}^2\text{-s}$$

C. Compute Effective Available Time–Space

The number of turning vehicles during the walk and pedestrian clear intervals is calculated with Equation 18-64.

$$N_{tv} = \frac{v_{lt,perm} + v_{rt} - v_{rtor}}{3{,}600} C$$

$$N_{tv} = \frac{42 + 76 - 38}{3{,}600}(80) = 1.8 \text{ veh}$$

The time–space occupied by turning vehicles can then be computed with Equation 18-63.

$$TS_{tv} = 40 N_{tv} W_c$$

$$TS_{tv} = 40(1.8)(16) = 1{,}138 \text{ ft}^2\text{-s}$$

The effective available crosswalk time–space TS_{cw}* is found by subtracting the total available crosswalk time–space TS_{cw} from the time–space occupied by turning vehicles.

$$TS_{cw}^{*} = TS_{cw} - TS_{tv}$$

$$TS_{cw}^{*} = 4{,}928 - 1{,}138 = 3{,}790 \text{ ft}^2\text{-s}$$

D. Compute Pedestrian Service Time

The number of pedestrians exiting the curb when the WALK indication is presented is as follows:

$$N_{ped,co} = N_{co} \frac{C - g_{\text{Walk},mj}}{C}$$

$$N_{ped,co} = (11.8)\frac{80 - 11}{80} = 10.2 \text{ p}$$

Because the crosswalk width is greater than 10 ft, the pedestrian service time is computed as follows:

$$t_{ps,co} = 3.2 + \frac{L_c}{S_p} + 2.7\frac{N_{ped,co}}{W_c}$$

$$t_{ps,co} = 3.2 + \frac{28}{4.0} + 2.7\left(\frac{10.2}{16}\right) = 11.9 \text{ s}$$

The other travel direction in the crosswalk is analyzed next. The number of pedestrians arriving at Corner 1 each cycle by crossing the minor street is as follows:

$$N_{ci} = \frac{v_{ci}}{3{,}600}C$$

$$N_{ci} = \frac{490}{3{,}600}(80) = 10.9 \text{ p}$$

The sequence of calculations is repeated for this second travel direction in the subject crosswalk to indicate that $N_{ped,ci}$ is equal to 9.4 p and $t_{ps,ci}$ is 11.8.

E. Compute Crosswalk Occupancy Time

The crosswalk occupancy time for the crosswalk is computed as follows:

$$T_{occ} = t_{ps,co} N_{co} + t_{ps,ci} N_{ci}$$

$$T_{occ} = 11.9\ (11.8) + 11.8\ (10.9) = 268.6 \text{ p-s}$$

F. Compute Pedestrian Crosswalk Circulation Area

Finally, the crosswalk circulation area per pedestrian for the crosswalk is computed as follows:

$$M_{cw} = \frac{TS^*_{cw}}{T_{occ}}$$

$$M_{cw} = \frac{3{,}790}{268.6} = 14.1 \text{ ft}^2/\text{p}$$

The crosswalk circulation area is found to be 14.1 ft^2/p. According to the qualitative descriptions provided in Exhibit 18-24, pedestrians will find that their walking speed is restricted, with very limited ability to pass slower pedestrians. Improvements to the crosswalk should be considered and may include a wider crosswalk or a longer walk interval.

Step 3: Determine Pedestrian Delay

The pedestrian delay is calculated as follows:

$$d_p = \frac{(C - g_{\text{Walk},mj})^2}{2\,C}$$

$$d_p = \frac{(80-11)^2}{2\,(80)} = 29.8 \text{ s/p}$$

Step 4: Determine Pedestrian LOS Score for Intersection

The number of vehicles traveling on the minor street during a 15-min period is computed as follows:

$$n_{15,mi} = \frac{0.25}{N_c} \sum v_i$$

$$n_{15,mi} = \frac{0.25}{2}(72+336+60+42+400+76) = 123.3 \text{ veh/ln}$$

The cross-section adjustment factor is calculated as follows:

$$F_w = 0.681(N_c)^{0.514}$$

$$F_w = 0.681(2)^{0.514} = 0.972$$

The motorized vehicle adjustment factor is computed as follows:

$$F_v = 0.00569\left(\frac{v_{rtor} + v_{lt,perm}}{4}\right) - N_{rtci,c}(0.0027 n_{15,mi} - 0.1946)$$

$$F_v = 0.00569\left(\frac{30+42}{4}\right) - (0)\,[0.0027(123.3) - 0.1946] = 0.102$$

The motorized vehicle speed adjustment factor is then computed:

$$F_s = 0.00013\; n_{15,mi} S_{85,mi}$$

$$F_s = 0.00013(123.3)(35) = 0.561$$

The pedestrian delay adjustment factor is calculated as follows:

$$F_{\text{delay}} = 0.0401 \ln(d_{p,c})$$

$$F_{\text{delay}} = 0.0401 \ln(29.8) = 0.136$$

The pedestrian LOS score for the intersection $I_{p,int}$ is then computed as follows:

$$I_{p,int} = 0.5997 + F_w + F_v + F_S + F_{\text{delay}}$$

$$I_{p,int} = 0.5997 + 0.972 + 0.102 + 0.561 + 0.136 = 2.37$$

For this crosswalk, $I_{p,int}$ is found to be 2.37.

Step 5: Determine LOS

According to Exhibit 18-5, the crosswalk operates at LOS B.

Discussion

The crosswalk was found to operate at LOS B in Step 5. It was determined in Step 1 that the pedestrians at both corners have adequate space to allow freedom of movement. Crosswalk circulation area was found to be restricted in Step 2 and improvements are probably justified. Moreover, the pedestrian delay computed in Step 3 was found to be slightly less than 30 s/p. With this much delay, some pedestrians may not comply with the signal indication.

EXAMPLE PROBLEM 3: BICYCLE LOS

The Intersection

A 5-ft-wide bicycle lane is provided at a signalized intersection.

The Question

What is the LOS of this bicycle lane?

The Facts

Saturation flow rate for bicycles = 2,000 bicycles/h

Effective green time = 48 s

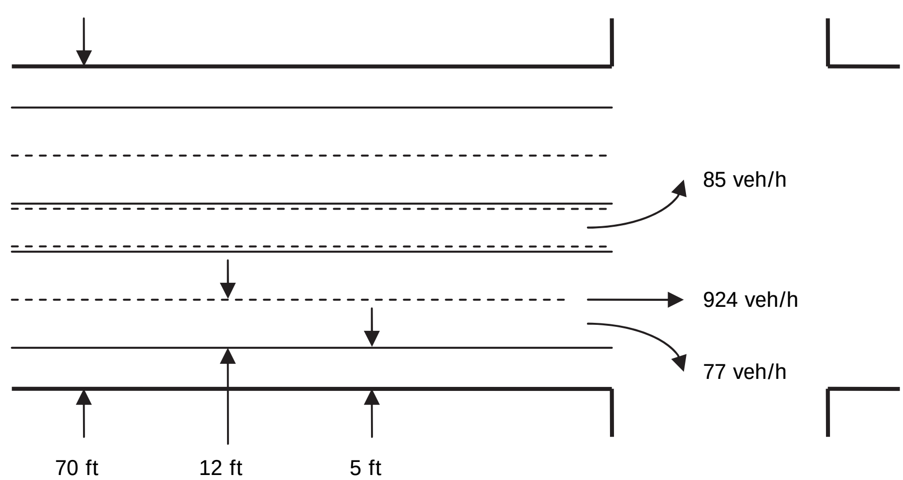
Cycle length = 120 s

Bicycle flow rate = 120 bicycles/h

No on-street parking

The vehicular flow rates and street cross-section element widths are as shown in Exhibit 18-49.

Exhibit 18-49
Example Problem 3: Vehicular Demand Flow Rates and Cross-Section Element Widths



Outline of Solution

Bicycle delay and the bicycle LOS score will be computed. LOS is then determined on the basis of the computed score and the threshold values in Exhibit 18-5.

Computational Steps

Step 1: Determine Bicycle Delay

A. Compute Bicycle Lane Capacity

The capacity of the bicycle lane is calculated with Equation 18-78:

$$c_b = s_b \frac{g_b}{C}$$

$$c_b = (2{,}000)\frac{48}{120} = 800 \text{ bicycles/h}$$

B. Compute Bicycle Delay

Bicycle delay is computed with Equation 18-79:

$$d_b = \frac{0.5C(1 - g_b / C)^2}{1 - \frac{g_b}{C}\mathrm{Min}\left[\frac{v_{bic}}{c_b}, 1.0\right]}$$

$$d_b = \frac{0.5(120)(1 - 48/120)^2}{1 - \frac{48}{120}\mathrm{Min}\left[\frac{120}{800}, 1.0\right]} = 23.0 \text{ s/bicycle}$$

Step 2: Determine Bicycle LOS Score for Intersection

As shown in Exhibit 18-49, the total width of the outside through lane, bicycle lane, and paved shoulder is 17 ft (= 12 + 5 + 0). The cross-section adjustment factor can then be calculated with Equation 18-81:

$$F_w = 0.0153W_{cd} - 0.2144W_t$$

$$F_w = 0.0153(70) - 0.2144(17) = -2.57$$

The motor-vehicle volume adjustment factor must also be calculated, by using Equation 18-82:

$$F_v = 0.0066 \frac{v_{lt} + v_{th} + v_{rt}}{4N_{th}}$$

$$F_v = 0.0066 \frac{85 + 924 + 77}{4(2)} = 0.90$$

The bicycle LOS score can then be computed with Equation 18-80:

$$I_{b,int} = 4.1324 + F_w + F_v$$

$$I_{b,int} = 4.1324 - 2.57 + 0.90 = 2.45$$

Step 3: Determine LOS

According to Exhibit 18-5, this bicycle lane would operate at LOS B through the signalized intersection.

Discussion

The bicycle lane was found to operate at LOS B. The bicycle delay was found to be 23.0 s/bicycle, which is low enough that most bicyclists are not likely to be impatient. However, if the signal timing at the intersection were to be changed, the bicycle delay would need to be computed again to verify that it does not rise above 30 s/bicycle.

5. REFERENCES

1. *Signalized Intersection Capacity Method.* NCHRP Project 3-28(2). JHK & Associates, Tucson, Ariz., Feb. 1983.
2. *Signalized Intersection Capacity Study: Final Report.* NCHRP Project 3-28(2). JHK & Associates, Tucson, Ariz., Dec. 1982.
3. Messer, C. J., and D. B. Fambro. Critical Lane Analysis for Intersection Design. In *Transportation Research Record 644*, Transportation Research Board, National Research Council, Washington, D.C., 1977, pp. 26–35.
4. Berry, D. S., and P. K. Gandhi. Headway Approach to Intersection Capacity. In *Highway Research Record 453,* Highway Research Board, National Research Council, Washington, D.C., 1973, pp. 56–60.
5. Miller, A. *Australian Road Research Bulletin 3: The Capacity of Signalized Intersections in Australia*. Australian Road Research Board, Kew, Victoria, 1968.
6. Webster, F., and B. Cobbe. *Traffic Signals.* Road Research Technical Paper 56. Her Majesty's Stationery Office, London, 1966.
7. Petersen, B., and E. Imre. *Swedish Capacity Manual.* National Swedish Road Administration, Stockholm, Feb. 1977.
8. Catling, I. A Time-Dependent Approach to Junction Delays. *Traffic Engineering and Control,* Vol. 18, No. 11, Nov. 1977, pp. 520–526.
9. Akçelik, R. *Australian Road Research Report 123: Traffic Signals: Capacity and Timing Analysis.* Australian Road Research Board, Kew, Victoria, 1981.
10. Reilly, W., C. Gardner, and J. Kell. *A Technique for Measurement of Delay at Intersections.* Report FHWA-RD-76-135/137. Federal Highway Administration, Washington, D.C., 1976.
11. Courage, K. G., D. B. Fambro, R. Akçelik, P.-S. Lin, M. Anwar, and F. Viloria. *NCHRP Web Document 10: Capacity Analysis of Traffic-Actuated Intersections.* Transportation Research Board, National Research Council, Washington, D.C., Dec. 1996.
12. Akçelik, R. Estimation of Green Times and Cycle Time for Vehicle-Actuated Signals. In *Transportation Research Record 1457,* Transportation Research Board, National Research Council, Washington, D.C., 1994, pp. 63–72.
13. Lin, F.-B. Estimation of Average Phase Durations for Full-Actuated Signals. In *Transportation Research Record 881,* Transportation Research Board, National Research Council, Washington, D.C., 1982, pp. 65–72.
14. Lin, F.-B. Predictive Models of Traffic-Actuated Cycle Splits. *Transportation Research*, Vol. 16B, No. 5, 1982, pp. 361–372.
15. Cowan, R. Useful Headway Models. *Transportation Research*, Vol. 9, No. 6, 1975, pp. 371–375.

16. Akçelik, R., and E. Chung. Calibration of the Bunched Exponential Distribution of Arrival Headways. *Road and Transport Research*, Vol. 3, No. 1, 1994, pp. 42–59.
17. Lin, F.-B. Optimal Timing Settings and Detector Lengths of Presence Mode Full-Actuated Control. In *Transportation Research Record 1010,* Transportation Research Board, National Research Council, Washington, D.C., 1985, pp. 37–45.
18. Milazzo, J. S., II, N. M. Rouphail, J. E. Hummer, and D. P. Allen. Effect of Pedestrians on Capacity of Signalized Intersections. In *Transportation Research Record 1646,* Transportation Research Board, National Research Council, Washington, D.C., 1998, pp. 37–46.
19. Allen, D. P., J. E. Hummer, N. M. Rouphail, and J. S. Milazzo II. Effect of Bicycles on Capacity of Signalized Intersections. In *Transportation Research Record 1646,* Transportation Research Board, National Research Council, Washington, D.C., 1998, pp. 87–95.
20. Rouphail, N., J. Hummer, J. Milazzo, and D. Allen. *Recommended Procedures for Chapter 9, Signalized Intersections, of the Highway Capacity Manual.* Report FHWA-RD-98-106. Federal Highway Administration, Washington, D.C., 1999.
21. Rouphail, N. M., K. G. Courage, and D. W. Strong. New Calculation Method for Existing and Extended *Highway Capacity Manual* Delay Estimation Procedures. Presented at 85th Annual Meeting of the Transportation Research Board, Washington, D.C., 2006.
22. Strong, D. W., and N. M. Rouphail. Incorporating Effects of Traffic Signal Progression into Proposed Incremental Queue Accumulation Method. Presented at 85th Annual Meeting of the Transportation Research Board, Washington, D.C., 2006.
23. Prassas, E. S., and R. P. Roess. Left-Turn Adjustment for Permitted Turns from Shared Lane Groups: Another Look. In *Transportation Research Record 1398,* Transportation Research Board, National Research Council, Washington, D.C., 1993, pp. 75–81.
24. Bonneson, J. A., M. P. Pratt, and M. A. Vandehey. *Predicting the Performance of Automobile Traffic on Urban Streets: Final Report.* National Cooperative Highway Research Program Project 3-79. Texas Transportation Institute, Texas A&M University, College Station, Jan. 2008.
25. Dowling, R. G., D. B. Reinke, A. Flannery, P. Ryus, M. Vandehey, T. A. Petritsch, B. W. Landis, N. M. Rouphail, and J. A. Bonneson. *NCHRP Report 616: Multimodal Level of Service Analysis for Urban Streets.* Transportation Research Board of the National Academies, Washington, D.C., 2008.
26. Rouphail, N., J. Hummer, J. Milazzo, and D. Allen. *Capacity Analysis of Pedestrian and Bicycle Facilities: Recommended Procedures for the "Pedestrians" Chapter of the Highway Capacity Manual.* Report FHWA-RD-98-107. Federal Highway Administration, Washington, D.C., 1998.

27. Rouphail, N., J. Hummer, J. Milazzo, and D. Allen. *Capacity Analysis of Pedestrian and Bicycle Facilities: Recommended Procedures for the "Bicycles" Chapter of the Highway Capacity Manual*. Federal Highway Administration, Washington, D.C., 1998.
28. Virkler, M. R. Pedestrian Compliance Effects on Signal Delay. In *Transportation Research Record 1636*, Transportation Research Board, National Research Council, Washington, D.C., 1998, pp. 88–91.
29. Virkler, M. R. Scramble and Crosswalk Signal Timing. In *Transportation Research Record 1636*, Transportation Research Board, National Research Council, Washington, D.C., 1998, pp. 83–87.
30. Dunn, R., and R. Pretty. Mid-Block Pedestrian Crossings—An Examination of Delay. *Proc., 12th Annual Australian Road Research Board Conference*, Hobart, Tasmania, Australia, Aug. 1984.
31. Opiela, K. S., S. Khasnabis, and T. K. Datta. Determination of the Characteristics of Bicycle Traffic at Urban Intersections. In *Transportation Research Record 743*, Transportation Research Board, National Research Council, Washington, D.C., 1980, pp. 30–38.
32. Pline, J. (ed.). *Traffic Control Devices Handbook.* Institute of Transportation Engineers, Washington, D.C., 2001.
33. Koonce, P., L. Rodegerdts, K. Lee, S. Quayle, S. Beaird, C. Braud, J. Bonneson, P. Tarnoff, and T. Urbanik. *Traffic Signal Timing Manual.* Report FHWA-HOP-08-024. Federal Highway Administration, Washington, D.C., 2008.
34. *Manual on Uniform Traffic Control Devices for Streets and Highways*. Federal Highway Administration, Washington, D.C., 2009.
35. Zegeer, J. D., M. A. Vandehey, M. Blogg, K. Nguyen, and M. Ereti. *NCHRP Report 599: Default Values for Highway Capacity and Level of Service Analyses.* Transportation Research Board of the National Academies, Washington, D.C., 2008.