Signalized Intersection

Introduction

A methodology for evaluating the capacity and quality of service provided to road users traveling through a signalized intersection 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. HCM 2010 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.

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.

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.

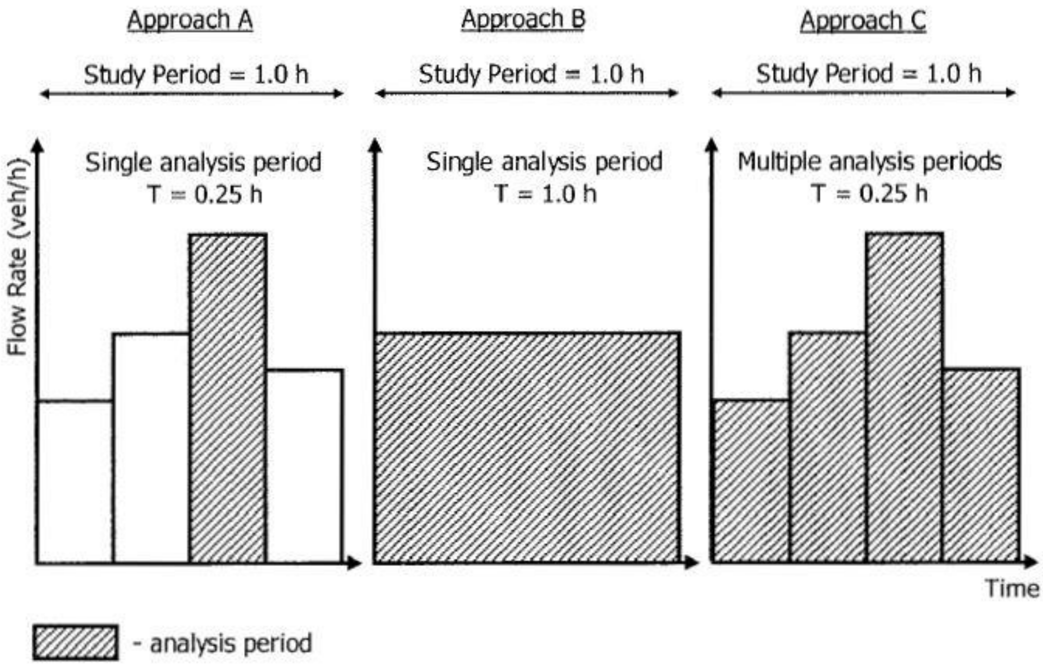
The methodology is based on the assumption that traffic conditions are steady during the analysis period (i.e., systematic change over time is negligible). For this reason, the analysis period ranges from 0.25 to 1h. The longer durations are sometimes used for planning analyses. In general, the analyst should use caution with analysis periods that exceed 1hbecause 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 18-1demonstrates 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 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-hdemand 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-hanalysis 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-hdemand 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-hstudy 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.

Travel Modes

HCM 2010 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 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.

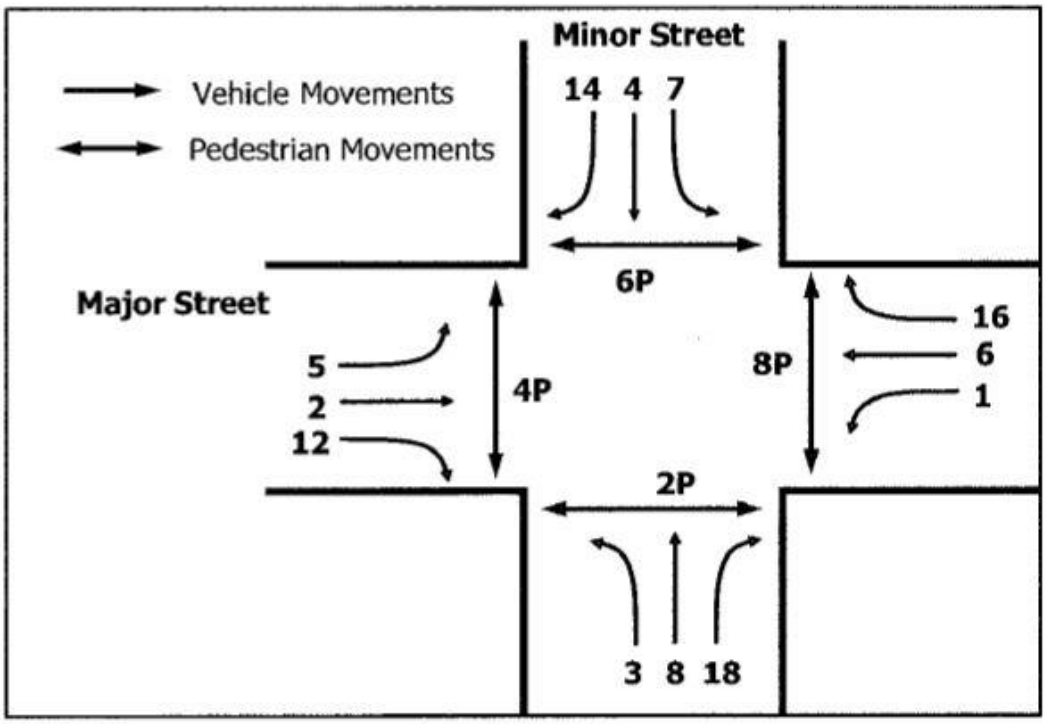
Lane Groups and Movement Groups

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.

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).

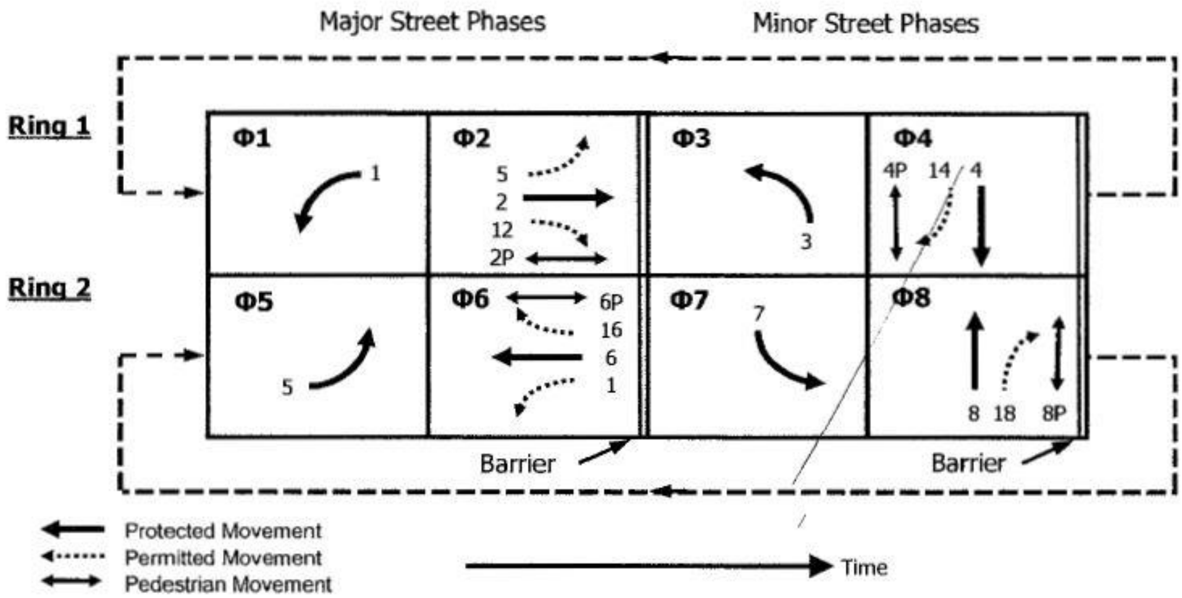
Exhibit 2 illustrates the vehicle and pedestrian traffic movements at a four-leg intersection. Three vehicular traffic movements and one pedestrian traffic movement are shown for each intersection approach. To facilitate the discussion in this chapter, each movement is assigned a unique number or a number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

<u>Exhibit 2</u> 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 3.

<u>Exhibit 3</u> Dual-Ring Structure with Illustrative Movement Assignments.



The symbol Ø shown in the exhibit above represents the word "phase," and the number following the symbol represents the phase number.

Exhibit 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.

LOS CRITERIA

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

Automobile Mode

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 between55 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 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.

Non automobile Modes

Exhibit 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 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

The input data needed for the automobile methodology. The data needed for fully or semi actuated signal control are listed in Exhibit 6. The additional data needed for coordinated-actuated control are listed in Exhibit 7. The last column of Exhibit 6 and Exhibit 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 6 and Exhibit 17 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 7 Input Data Requirements: Automobile Mode with Coordinated-Actuated 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
	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.

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

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 1h. 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-hperiod 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 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.

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 1h. 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:
Equation 1

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

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 1must be taken during a 15-minperiod that occurs within the 1-hperiod 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-minperiod 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-minanalysis period. If peak hour factors for each individual approach or movement are used, they are likely to generate demand volumes from one 15-minperiod that are in apparent conflict with demand volumes from another 15-minperiod, 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-mincount data are likely available, permitting the determination of actual 15-mindemand 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 8 provides an indication of the quality of progression associated with selected platoon ratio values.

Exhibit 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 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 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 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 2 are not known from field data, then the platoon ratio can be judged from Exhibit 8 by using the arrival type designation. Values of arrival

type range from 1to 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 containing40% 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 non-isolated intersections. The following equation is used to compute I for non-isolated intersections: equation 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 Xu 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,, can be approximated as the volume-to-capacity ratio of the contributing through movement at the upstream signalized intersection. The value of X,, used in Equation 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 12ft 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).

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 4.

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

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

<u>Number of Lanes</u>
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.

<u>Average Lane Width</u>
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 6 and Exhibit 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.

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 pushbuttons, 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 1or 2 s.

Cycle Length (Coordinated-Actuated Operation)

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

Analysis Period Duration

The analysis period is the time interval considered for the performance evaluation. It ranges from 15 min to 1h, with longer durations in this range sometimes used for planning analyses. In general, the analyst should interpret the results from an analysis period of 1hor 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.

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-minperiod of interest can be evaluated with the methodology; however, a complete evaluation should always include an analysis of conditions during the 15-minperiod that experiences the highest traffic demand during a 24-h period.

If traffic demand exceeds capacity for a given 15-minanalysis period, then a multiple-period analysis should be conducted. This type of analysis consists of an evaluation of several consecutive 15-mintime 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-minanalysis period is used for most planning analyses. However, hourly traffic demands are normally produced through the planning process. Thus, when 15-minforecast demands are not available for a 15-minanalysis period, a peak hour factor must be used to estimate the 15-min demands for the analysis period. A 1-hanalysis 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 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).

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.

Exhibit 9 Input Data Requirements: Non automobile 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

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.

<u>Permitted Left-Turn Flow Rate</u>

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 1h.A permitted left-turn movement can occur with either the permitted or the protected-permitted left-turn mode. For left-turn

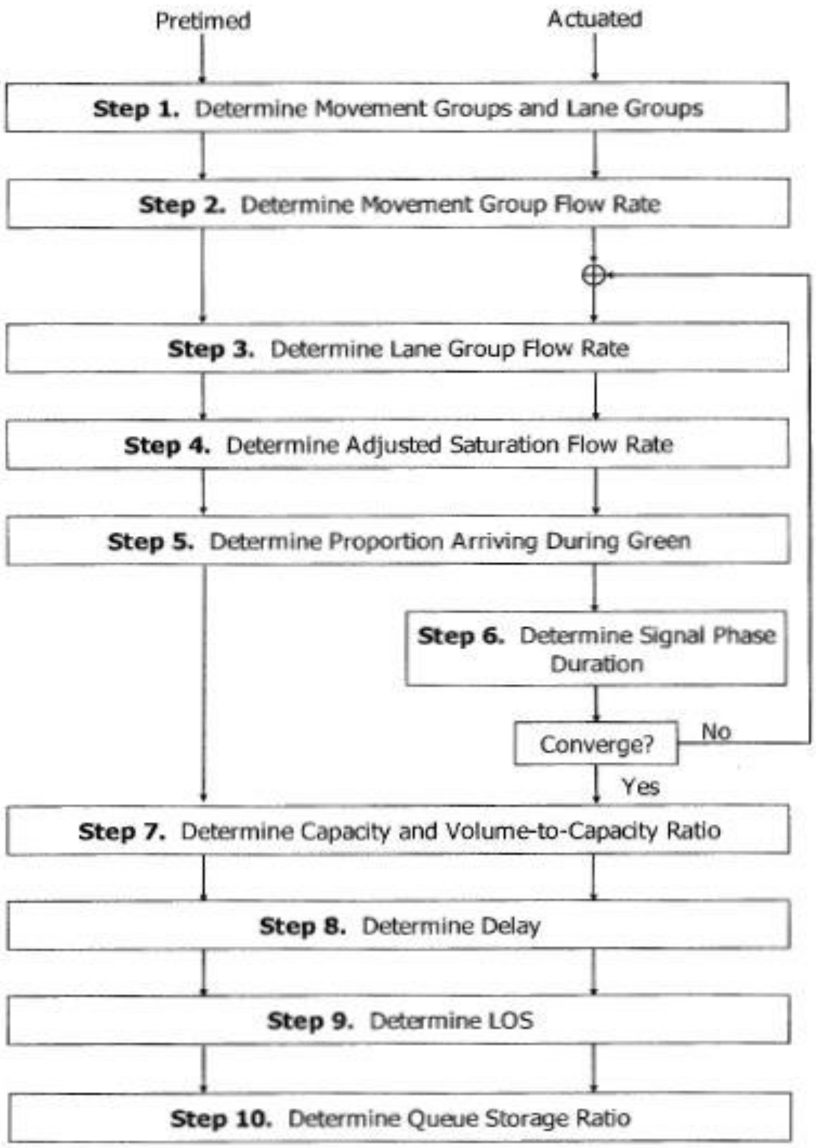
movements served by the permitted mode, the permitted left-turn flow rate is equal to the left-turn demand flow rate.
For left-turn movements served by the protected-permitted mode, the permitted left-turn flow rate should be measured in the field because its value is influenced by many factors.

Mid segment 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 1h. This flow rate is provided for each of five movements at each intersection corner.

Cycle Length
Cycle length is predetermined for pretimed or coordinated-actuated control.

Exhibit 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.



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.

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.

Exhibit 12 shows some common movement groups and lane groups.

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

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.

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

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

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.

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.

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 13.

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 maybe 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 16.

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

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 7.

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

Where Pg 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 8.

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

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 off,, 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_u 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 9.

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

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

Adjustment for Area Type

The area type adjustment factor /„ 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 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).

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 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).

Adjustment for Pedestrians and Bicycles

The procedure to determine the left-turn pedestrian-bicycle adjustment factor f_{Rpb} 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.

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 12 is used to compute this proportion for each lane group.

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

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.

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 13.

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

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).

Exhibit 14 Time Elements Influencing Actuated Phase Duration.

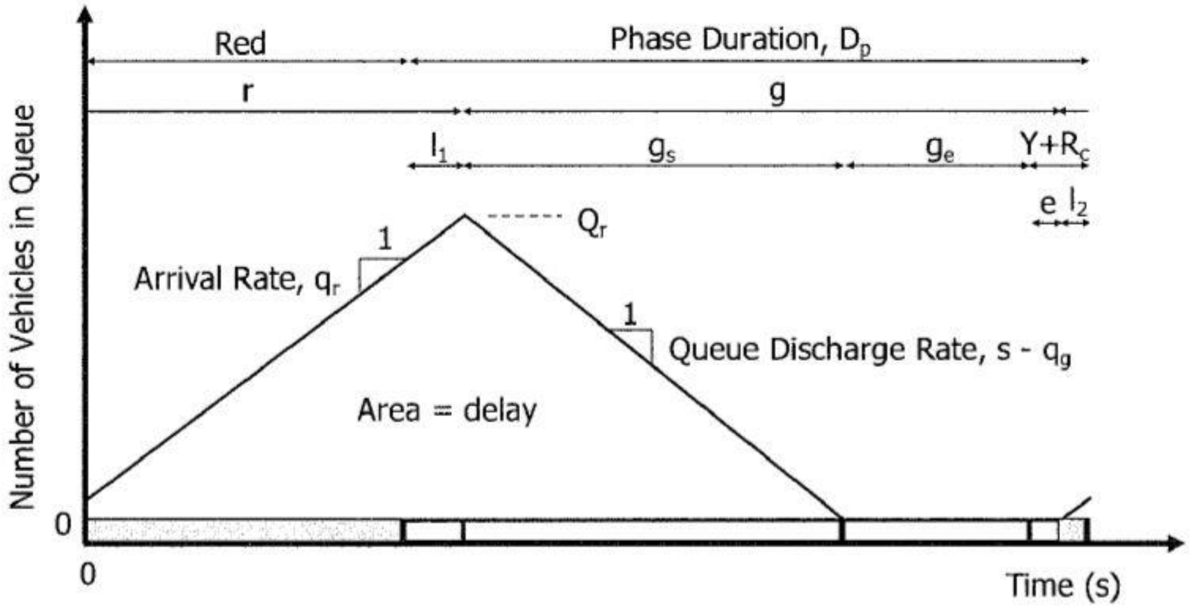


Exhibit 14 shows the relationship between phase duration and queue size for the average signal cycle. During the red interval, vehicles arrive at a rate of q_r and form a queue. The queue reaches its maximum size Z_j seconds after the red interval ends. At this time, the queue begins to discharge at a rate equal to the saturation flow rate s less the arrival rate during greenq_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 timeg_e.

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

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

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 15.

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

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.

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 16.

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

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 Xc. This ratio is computed by using Equation 17 with Equation 18.

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

with

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

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 3.

Equation 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 $y_{c,i}/\sum y_{c,i}$.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.

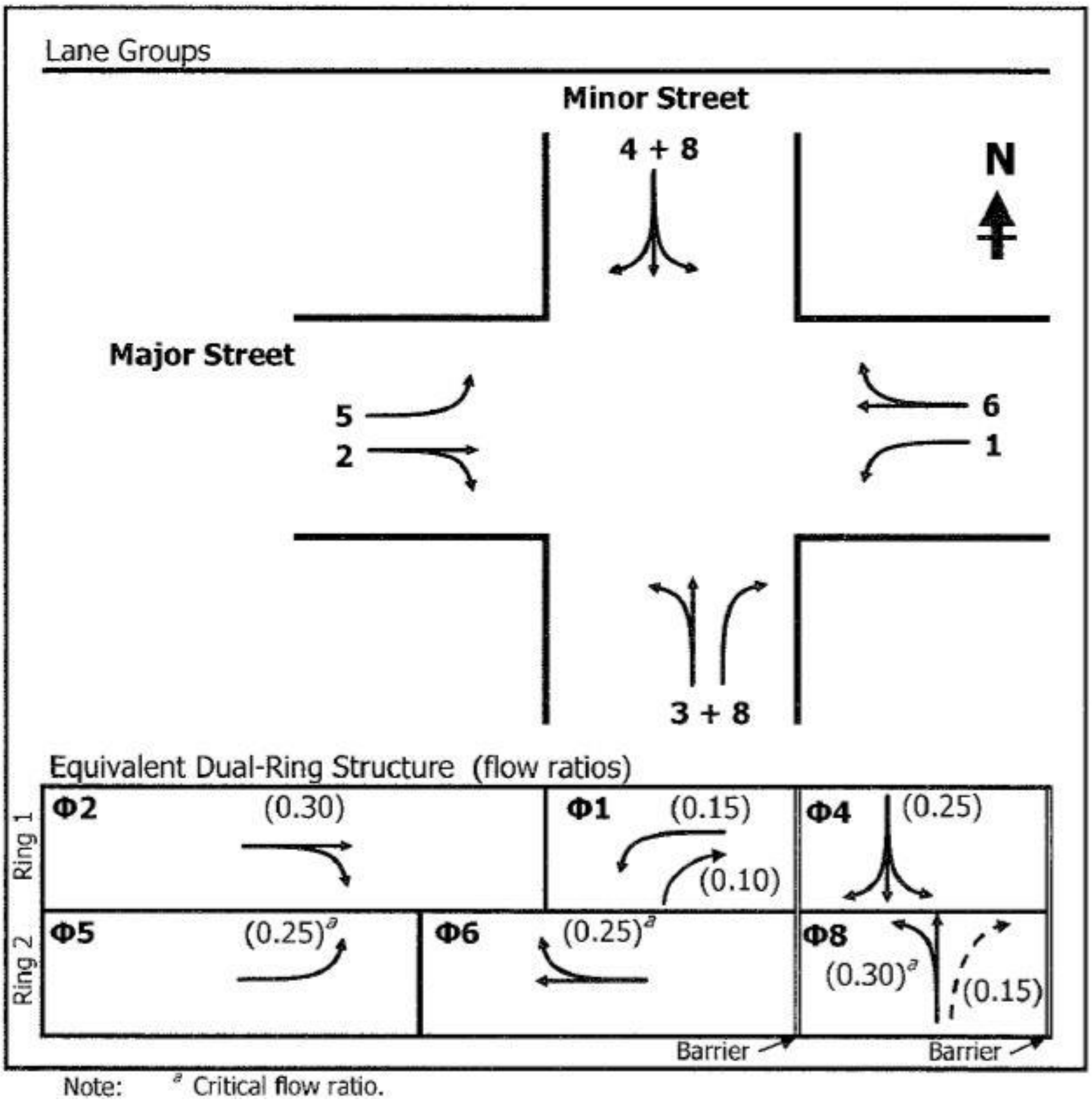
Next, the lane group flow ratio is computed for each lane group served by the phase. If a lane group is served only during one pretimed phase, then its flow ratio is computed as the lane group flow rate (per lane) divided by the lane group saturation flow rate [i.e.,v_i/Ns_i]. If a lane group is served 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 1and proceeds to the pair in Ring2. 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
Equation17. The lost time associated with each of the critical phases is added to yield the cycle lost time L.

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.



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.

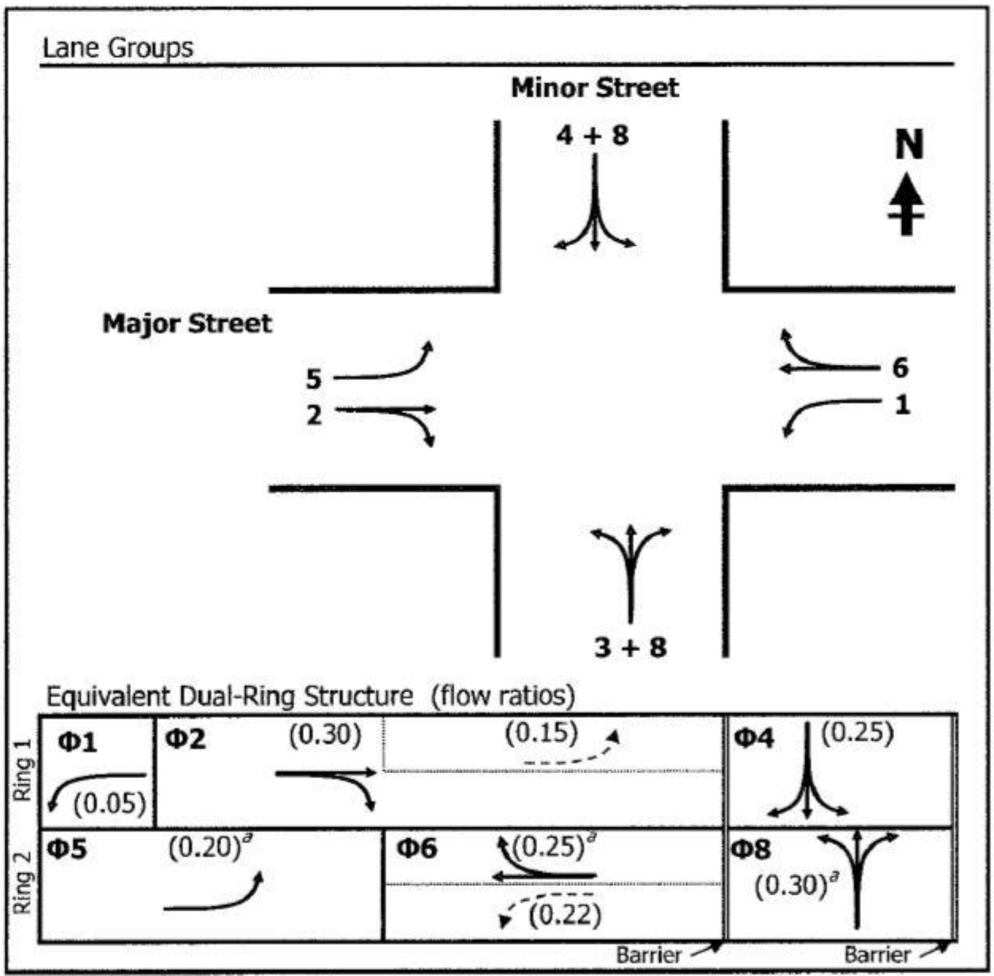
Phases 1, 2, 5, and 6 represent the phases between the other barrier pair. They serve the major-street approaches. A flow ratio is shown for the right-turn lane group in Phase 1because the intersection has pretimed control. If the intersection was actuated, it is unlikely that the right-turn detection 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 1is 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 1and 2 (i.e., Ring 1), and the other path is associated with Phases 5 and 6 (i.e., Ring2). The total phase flow ratio for the Ring 1path is 0.30 + 0.15, or 0.45. The total phase flow ratio for the Ring2 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 Z, 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 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 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 16. The following rules define the possible critical paths through this phase sequence:

1. One path is associated with Phases 1and 2 in Ring 1(0.35 =0.05 + 0.30).
2. One path is associated with Phases 5 and 6 inRing2 (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 1were shown to lag Phase 2 in Exhibit 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 inRing2 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 l2 associated with the first critical phase and the clearance lost time l2 associated with the second critical phase. If Rule 1, 2, or 4 applies, then one increment of phase lost time lt 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 1of Exhibit 15 and the treatment is the same. That is, both flow ratios are considered in defining the phase flow ratio for Phase 6.

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 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).

Uniform Delay

Equation 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.

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

All variables are as previously defined. The delay calculation procedure used in this methodology is consistent with Equation 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.

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 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 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 21 illustrates this calculation for interval i.

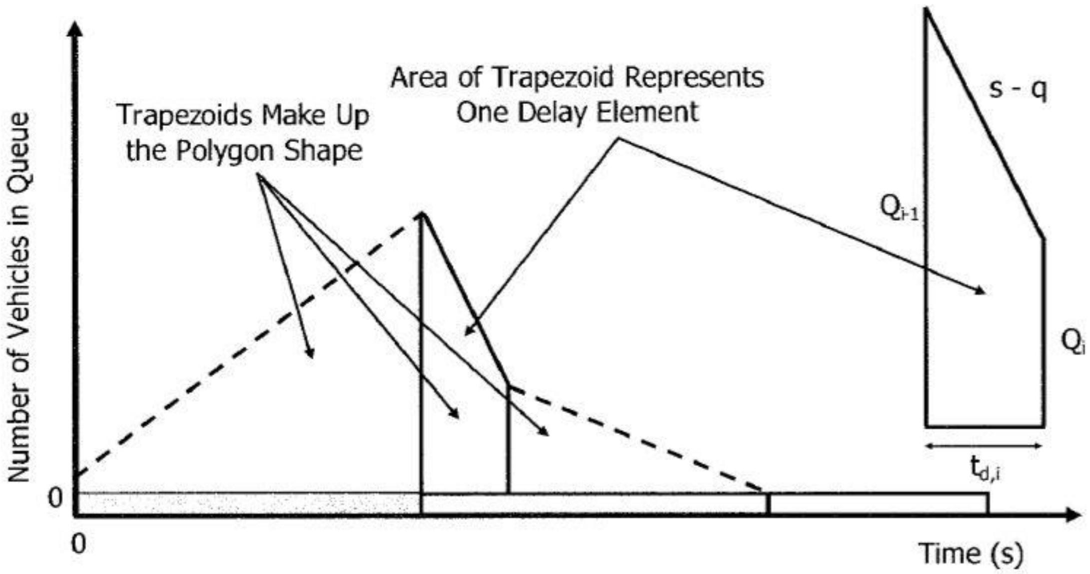


Exhibit 17 Decomposition of Queue Accumulation Polygon.

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

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.

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

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

Where d_{Ti} 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 (= q_C) 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:

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

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

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.

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 d2i in Exhibit 18.

<u>Exhibit18</u> Cumulative Arrivals and Departures during an Oversaturated Analysis Period

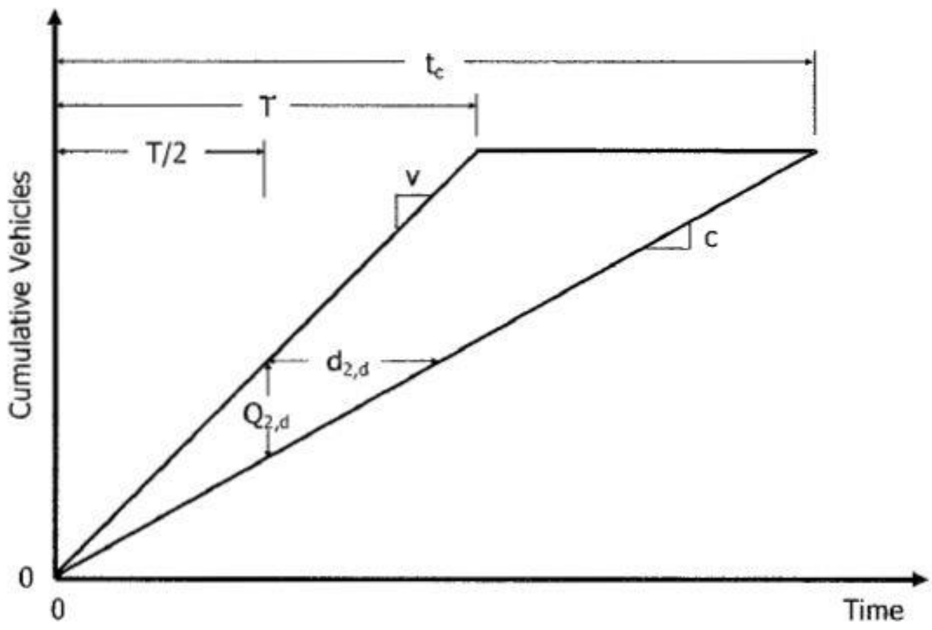


Exhibit 18-18 illustrates the queue growth that occurs as vehicles arrive at a demand flow rate v during analysis period T, which has capacity c. The deterministic delay component is represented by the triangular area bounded by the thick line and is associated with an average delay per vehicle represented 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.

<u>Initial Queue Delay</u>

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 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 variabled_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 19. However, it can be less than the analysis period duration for some lower-volume conditions.

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

Exhibit 19 Initial Queue Delay with Increasing Queue Size.

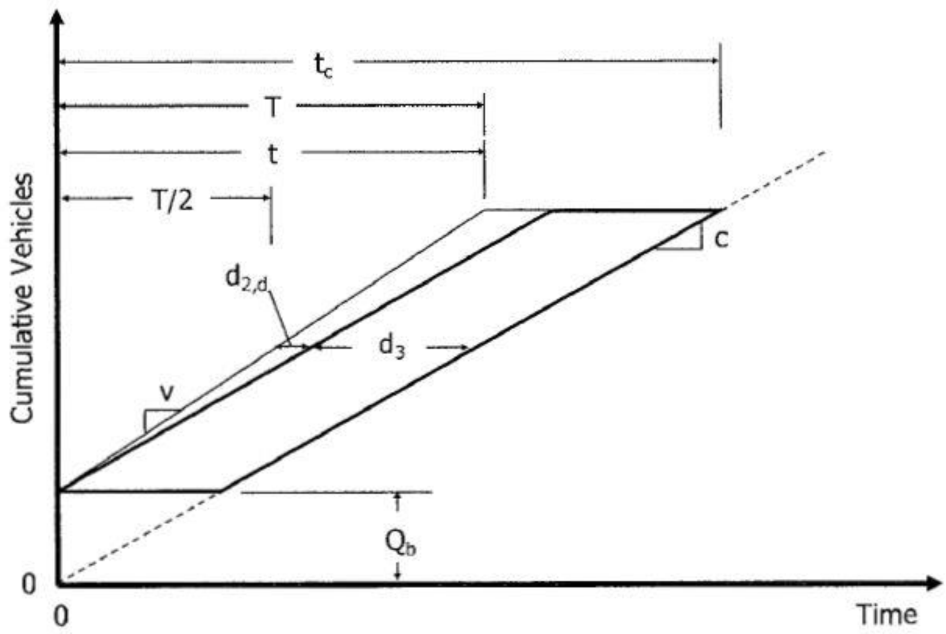


Exhibit 20 Initial Queue Delay with Decreasing Queue Size.

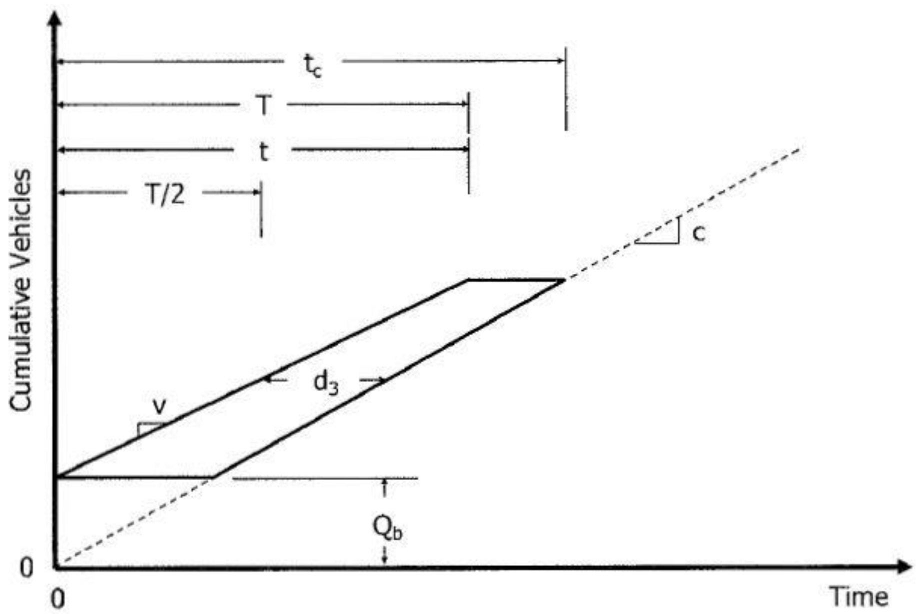
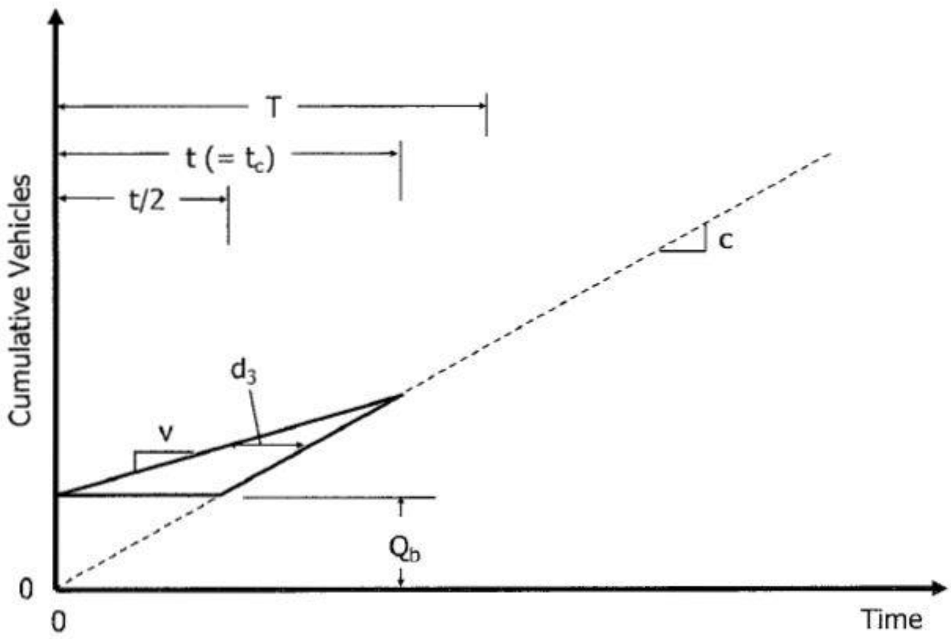


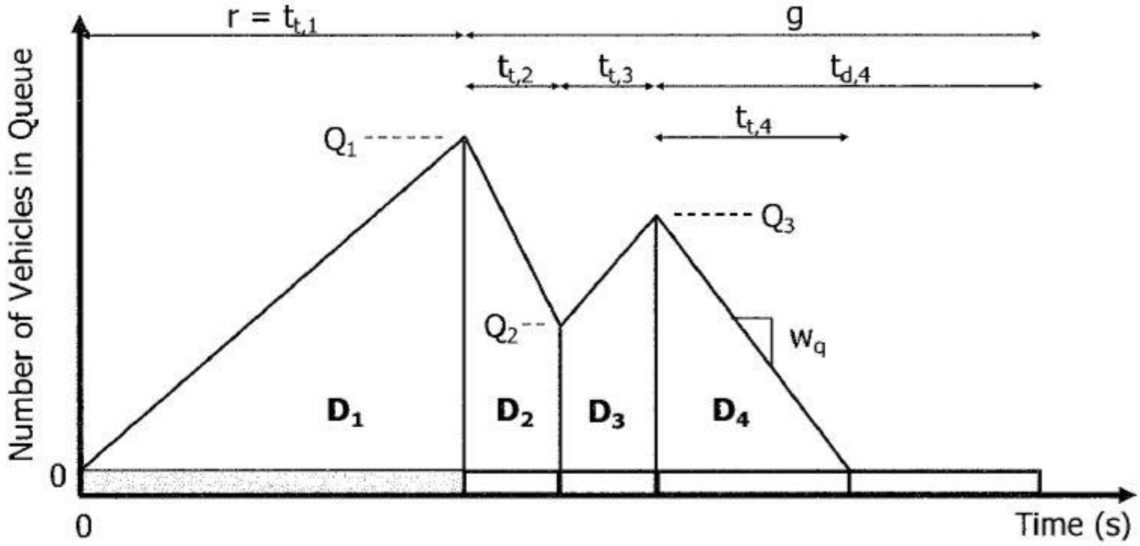
Exhibit 21 Initial Queue Delay with Queue Clearing.



A. Compute Baseline Uniform Delay

Exhibit 14was 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 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 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 25, with Equation 26.

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

with

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

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

The summation term in Equation 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 Interval4 in Exhibit 22.

B. Initial Queue Analysis

When a residual queue from a previous time period causes an initial queue to occur at the start of the analysis period (T), additional delay is experienced by vehicles arriving in the period since the initial queue must first clear the intersection. A procedure for determining this initial queue delay is described in detail in Appendix F. This procedure is also extended to analyze delay over multiple time periods, each having a duration T, in which an unmet demand may be carried from one time period to the next. If this is not the case, a value of zero is used for d_3.

Aggregated Delay Estimates

The procedure for delay estimation yields the control delay per vehicle for each lane group. It is often desirable to aggregate these values to provide delay for an intersection approach and for the intersection as a whole. This aggregation is done by computing weighted averages, where the lane group delays are weighted by the adjusted flows in the lane groups.

Thus, the delay for an approach is computed using Equation below:

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

where

d_A = delay for Approach A (s/veh),
d_i = delay for lane group i (on Approach A) (s/veh), and
v_i = adjusted flow for lane group i (veh/h).

Control delays on the approaches can be further aggregated using Equation 16-14 to provide the average control delay for the intersection:

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

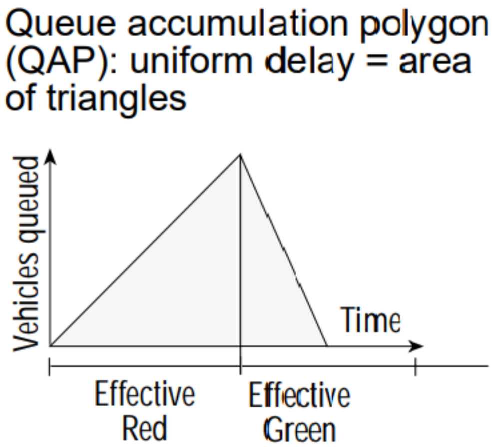
where

d_I = delay per vehicle for intersection (s/veh),
d_A = delay for Approach A (s/veh), and
v_A = adjusted flow for Approach A (veh/h).

Special Procedure for Uniform Delay with Protected-Plus-Permitted Left-Turn Operation from Exclusive Lanes

The first term in the delay calculation is easily derived as a function of the area contained within the plot of queue storage as a function of time. With a single green phase per cycle, this plot assumes a triangular shape; that is, the queue size increases linearly on the red phase and decreases linearly on the green. The peak storage occurs at the end of the red phase. The geometry of the triangle depends on the arrival flow rate, the queue discharge rate, and the length of the red and green signal phases.

This simple triangle becomes a more complex polygon when left turns are allowed to proceed on both protected and permitted phases. However, the area of this polygon, which determines the uniform delay, is still relatively easy to compute when the left turns are in an exclusive lane and the proper values for the arrival and discharge rates during the various intervals of the cycle are given along with the interval lengths that determine its shape. The procedure for this analysis is covered in Appendix E.



Determining Level of Service

Intersection LOS is directly related to the average control delay per vehicle. Once delays have been estimated for each lane group and aggregated for each approach and the intersection as a whole, Exhibit 16-2 is consulted, and the appropriate LOS is determined. The results of an operational application of this method will yield two key outputs: volume to capacity ratios for each lane group and for all of the critical lane groups within the intersection as a whole, and average control delays for each lane group and approach and for the intersection as a whole along with corresponding LOS.

Any v/c ratio greater than 1.0 is an indication of actual or potential breakdown. In such cases, multi-period analyses are advised. These analyses encompass all periods in which queue carryover due to oversaturation occurs. When the overall intersection v/c ratio is less than 1.0 but some critical lane groups have v/c ratios greater than 1.0, the green time is generally not appropriately apportioned, and a retiming using the existing phasing should be attempted. Appendix B should be consulted for guidelines.

A critical v/c ratio greater than 1.0 indicates that the overall signal and geometric design provides inadequate capacity for the given flows. Improvements that might be considered include basic changes in intersection geometry (number and use of lanes), increases in the signal cycle length if it is determined to be too short, and changes in the signal phase plan. Chapter 10 and Appendix B contain information on these types of improvements. Existing state and local policies or standards should also be consulted in the development of potential improvements.

LOS is a measure of the delay incurred by motorists at a signalized intersection. In some cases, delay will be high even when v/c ratios are low. In these situations, poor progression or an inappropriately long cycle length, or both, is generally the cause. Thus, an intersection can have unacceptably high delays without there being a capacity problem. When the v/c approaches or exceeds 1.0, it is possible that delay will remain at acceptable levels. This situation can occur, especially if the time over which high v/c levels occur is short. It can also occur if the analysis is for only a single period and there is queue carryover. In the latter case, conduct of a multi-period analysis is necessary to gain a true picture of delay. The analysis must consider the results of both the capacity analysis and the LOS analysis to obtain a complete picture of existing or projected intersection operations.

Determine back of Queue

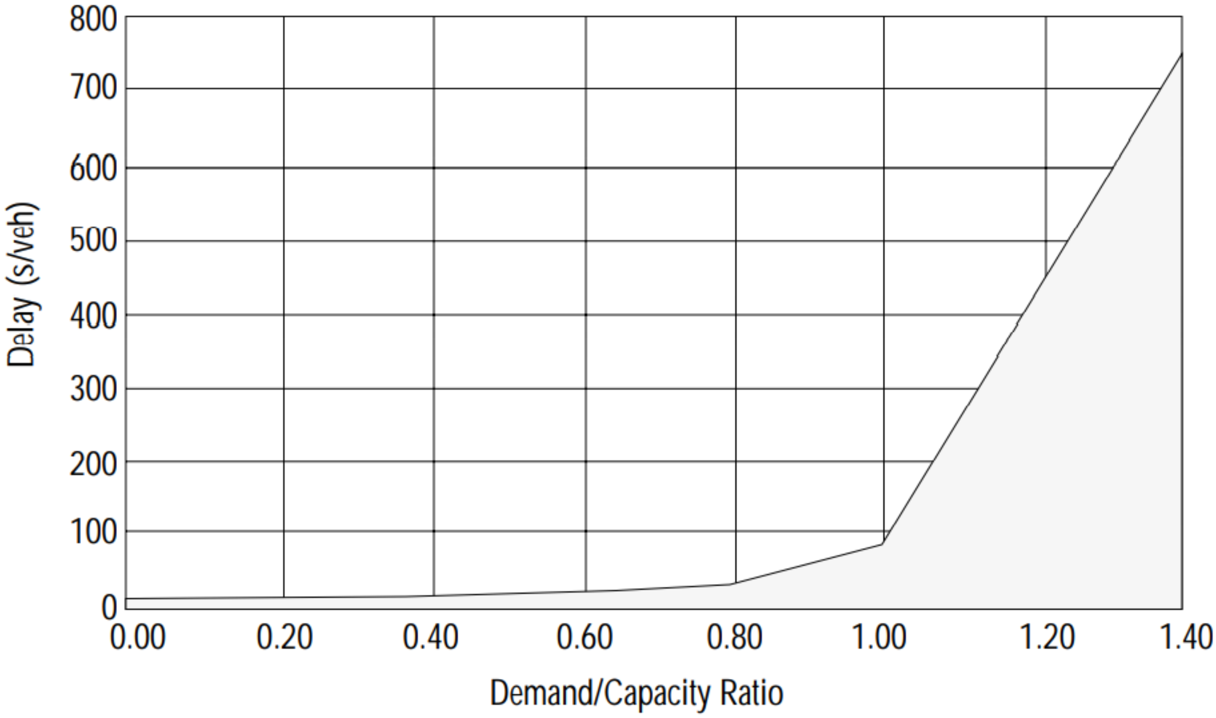
When an estimate of queue length is needed, a procedure to calculate the average back of queue and 70th-, 85th-, 90th-, and 98th-percentile back of queue is presented in Appendix G. The back of queue is the number of vehicles that are queued depending on the arrival patterns of vehicles and on the number of vehicles that do not clear the intersection during a given green phase (overflow). This procedure is also able to analyze back of queue over multiple time periods, each having a duration (T) in which an overflow queue may be carried from one time period to the next.

SENSITIVITY OF RESULTS TO INPUT VARIABLES

The methodology is sensitive to the geometric, demand, and control characteristics of the intersection. The predicted delay is highly sensitive to signal control characteristics and the quality of progression. The predicted delay is sensitive to the estimated saturation flow only when demand approaches or exceeds 90 percent of the capacity for a lane group or an intersection approach.

Exhibits 16-14 through 16-17 illustrate the sensitivity of the predicted control delay per vehicle to demand to capacity ratio, g/C, cycle length, and length of the analysis period (T). Delay is relatively insensitive to demand levels until demand exceeds 90 percent of capacity; then it is highly sensitive not only to changes in demand but also to changes in g/C, cycle length, and length of the analysis period. Initial queue delay, d_3, although not shown in Exhibit 16-14, occurs when there is queue spillback.

EXHIBIT 16-14. SENSITIVITY OF DELAY TO DEMAND TO CAPACITY RATIO
(SEE FOOTNOTE FOR ASSUMED VALUES)



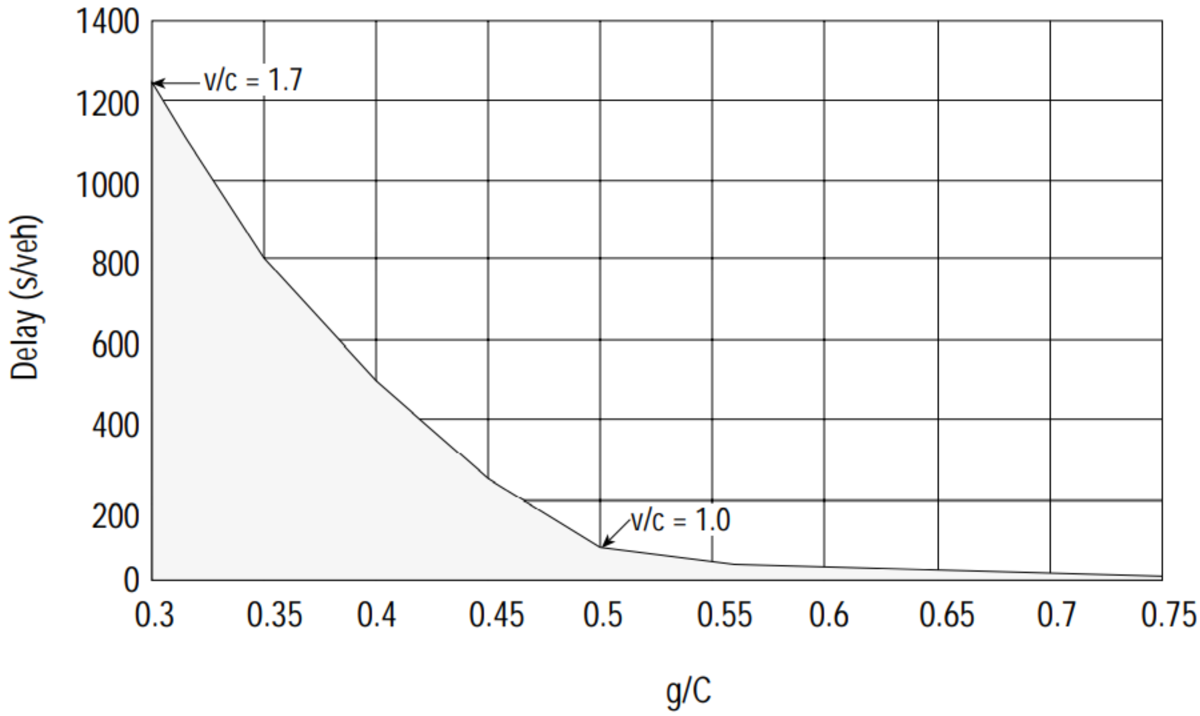
Note:
Assumptions: cycle length = 100 s , g/C = 0.5 , T = 1 h, k = 0.5, I = 1, s = 1800 veh/h.

Delay becomes sensitive to signal control parameters (cycle length, g/C, and progression) only at demand levels above 80 percent of capacity. Once demand exceeds 80 percent of capacity, modest increases in demand can cause significant increases in delay. The demand to capacity ratio itself is sensitive to the demand level, the PHF, the saturation flow rate, and the g/C ratio.

Small g/C values that do not provide sufficient capacity to serve the demand cause excessive delays for the movement. Once there is sufficient g/C to serve the movement, little is gained by providing more g/C to the movement (see Exhibit 16-15).

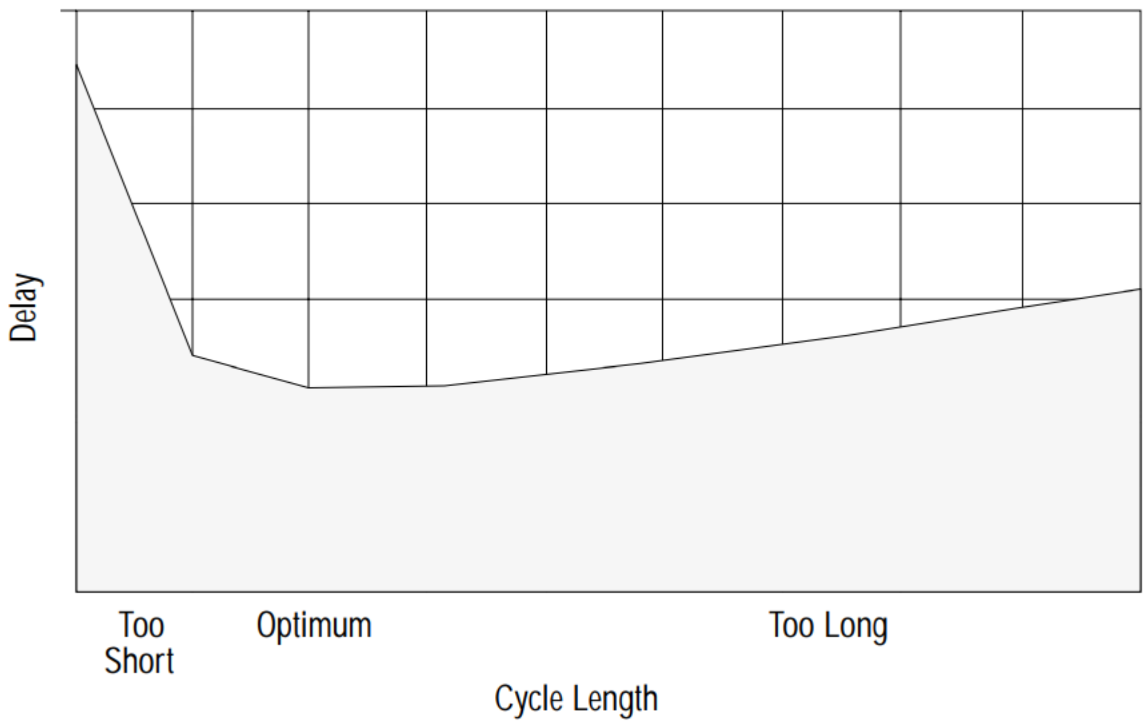
If the cycle length does not allow enough g/C time (which affects capacity) to serve a movement, the delay increases rapidly. Long cycle lengths also increase delay, but not as rapidly as short cycle lengths that provide insufficient capacity to serve the movements at the intersection (see Exhibit 16-16).

EXHIBIT 16-15. SENSITIVITY OF DELAY TO g/C
(SEE FOOTNOTE FOR ASSUMED VALUES)



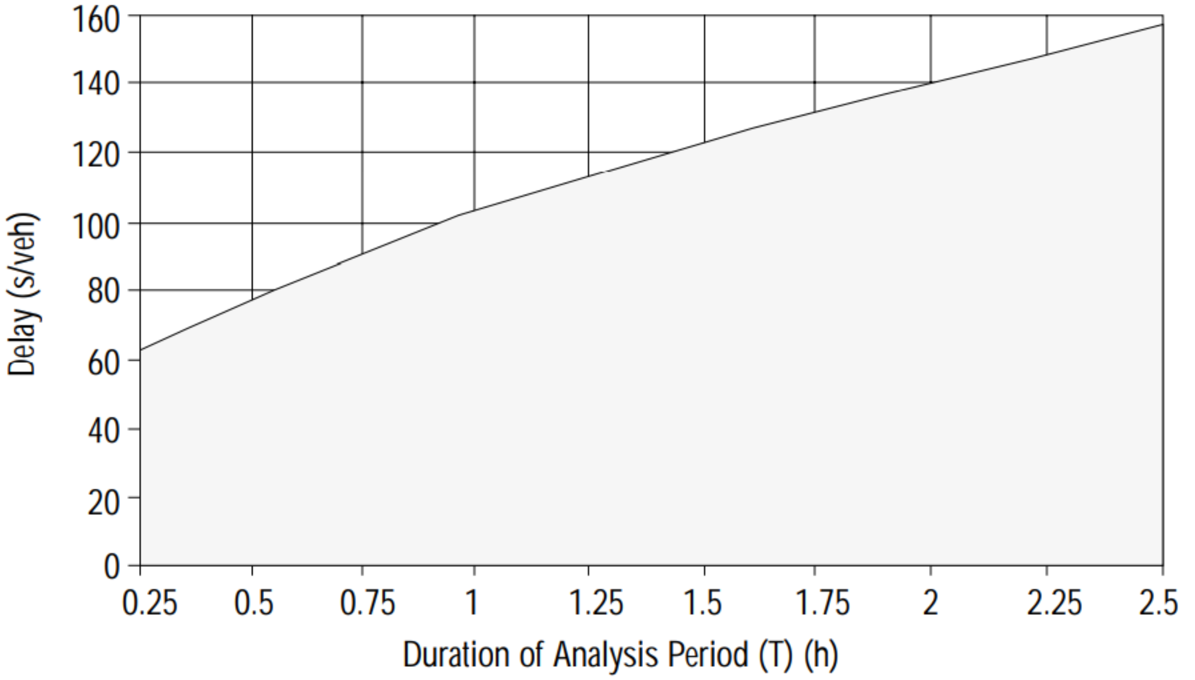
Note:
Assumptions: cycle length = 100 s , v/s = 0.5 , T = 1 h, k = 0.5, I = 1, s = 1800 veh/h.

EXHIBIT 16-16. SENSITIVITY OF DELAY TO CYCLE LENGTH



The length of the analysis period (T) determines how long the demand is assumed to be at the specified flow rate. When demand is less than capacity, the length of the analysis period has little influence on the estimated mean delay. However, when demand exceeds capacity, the longer analysis period means that a longer queue is built up and that it takes longer to clear the bottleneck. The result is that mean delay in oversaturated conditions is highly sensitive to the selected length of the analysis period (see Exhibit 16-17).

EXHIBIT 16-17. SENSITIVITY OF DELAY TO ANALYSIS PERIOD (T) (for v/c ≈ 1.0) (SEE FOOTNOTE FOR ASSUMED VALUES)



Note:
Assumptions: cycle length = 100 s , g/C = 0.4, v/s = 0.44 , k = 0.5, I = 1, s = 1800 veh/h.